Seismic Bridge Design Applications: Part One   Publication No. FHWA-SA-97-017

NHI Course No. 13063

Seismic Bridge Design Applications

25 April 1996

Part One

Publication No. FHWA-SA-97-017

Technical Report Documentation Page

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16. Abstract
Seismic Bridge Design Applications, Parts One and Two, contains the material used in two one-day national satellite seminars broadcast from the University of Maryland to provide seismic design application instruction. Mr. Robert Mast and Dr. Lee Marsh of BERGER/ABAM Engineers, Inc., were the instructors and developed the course materials. Part One includes seven sessions covering basic seismic principles, one complete seismic analysis and design example, modeling guidelines, multimodal analysis, and column design features. Part Two includes "homework problems" assigned after the first seminar as well as specific topics requested by participants of the first seminar.

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Seismic Design of Bridges, Seminar No.1 – Outline

Type of Material	Session No.	Topic
Background	1	Seismic Design Philosophy Seismic Hazard Analysis
	2	Structural Dynamics Response Spectra Overview of Division I-A
Worked Example	3	Two-Span Example Analysis
	4	Two-Span Example Design
Detailed Topics	5	Modeling Guidelines Foundation Modeling Multimode Analysis
	6	Multimode Analysis
	7	Intended Inelastic Behavior SPC B vs. SPC C and D Wall Pier Design Detailing Issues Questions and Answers

Session 1
Seismic Design Philosophy

- Code in 1973
- Lessons from Earthquakes
- Overall Objectives

AASHTO Through 1973

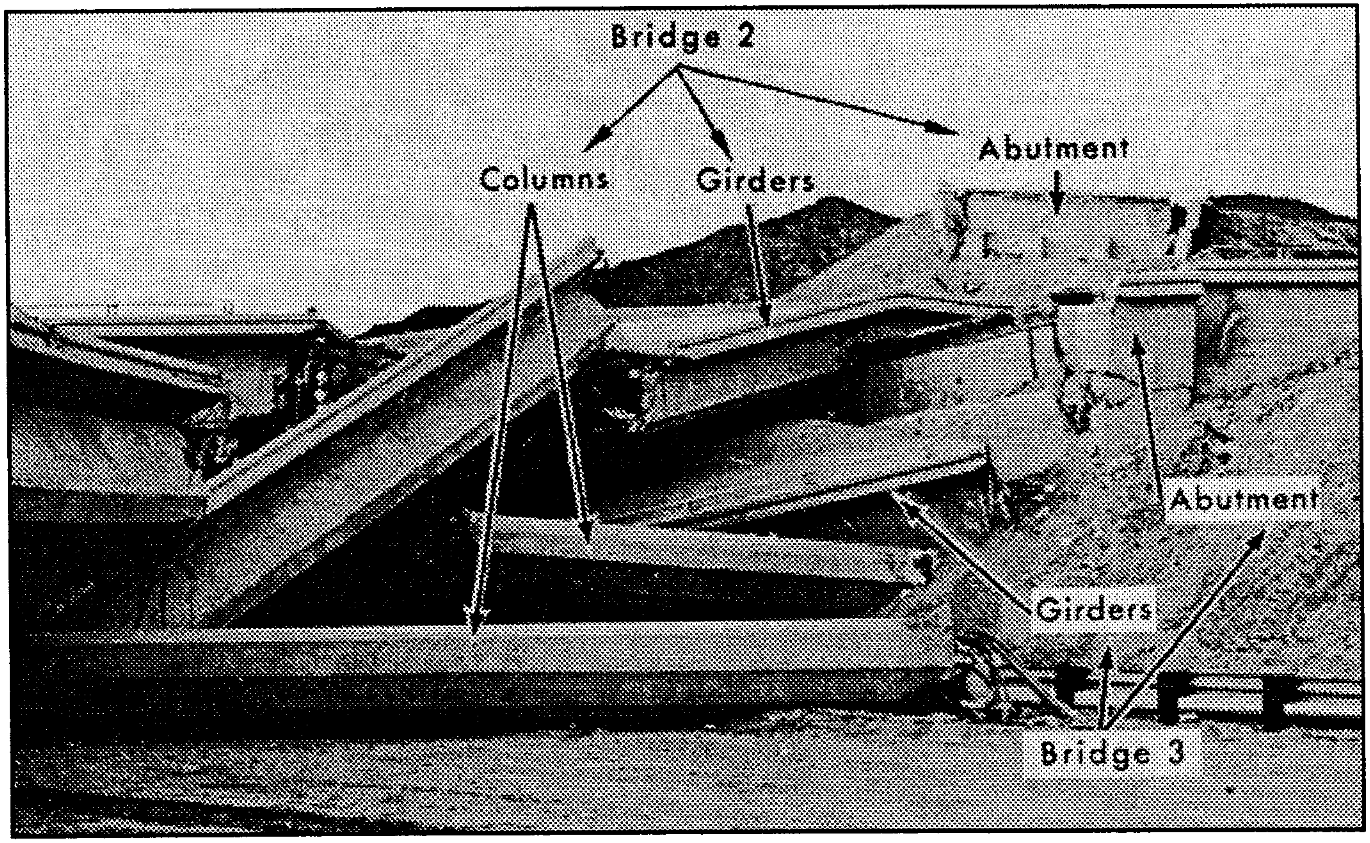
1.2.20 — Earthquake Stresses

In regions where earthquakes may be anticipated,

$$EQ = CD$$

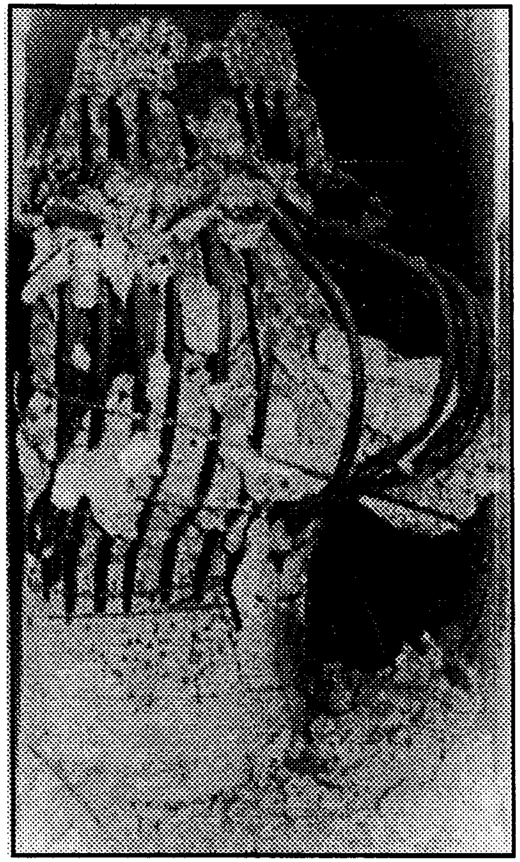
EQ = Lateral Force Applied Horizontally
D = Dead Load of Structure
C = 0.02 for Structures on Material Rated as 4 Tons or More per Square Foot
= 0.04 for Structures on Material Rated as Less than 4 Tons per Square Foot
= 0.06 for Structures on Piles

1971 San Fernando, CA



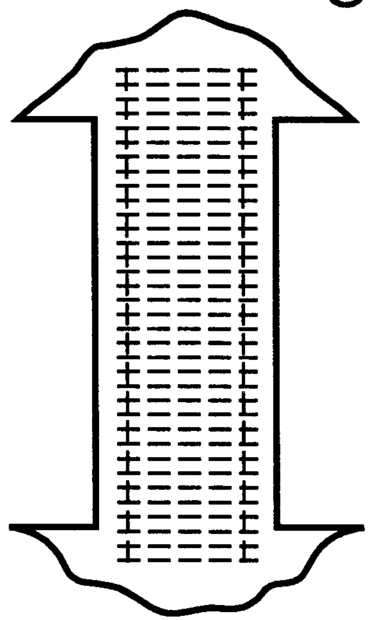
NBS

Failure Type: Shear



Nature: Brittle

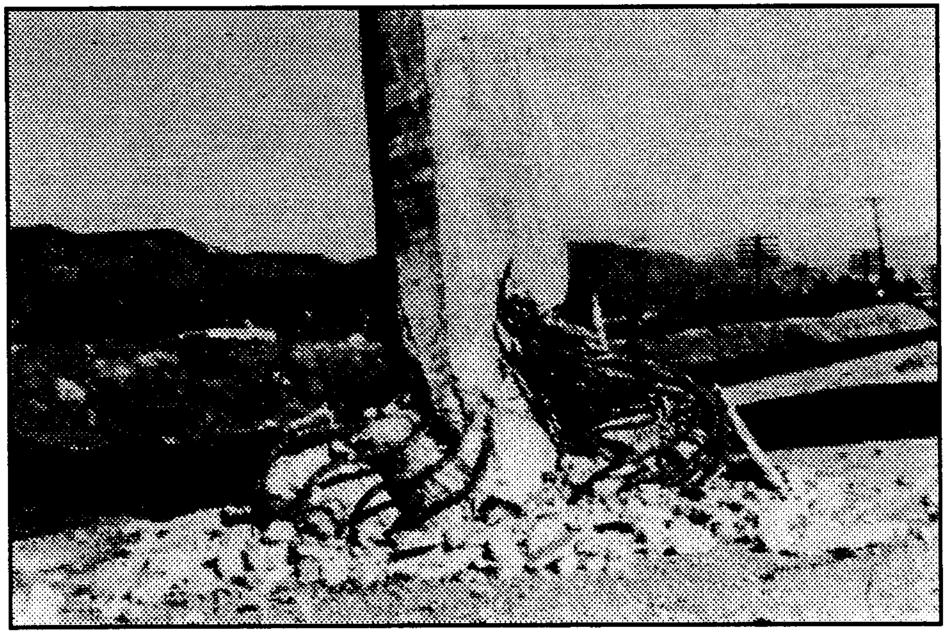
Prevention: Sufficient Shear Strength

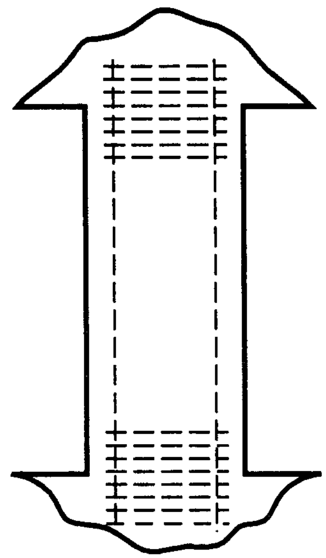


Failure: Bursting of Confinement

(Some Hinging then Shear Failure)

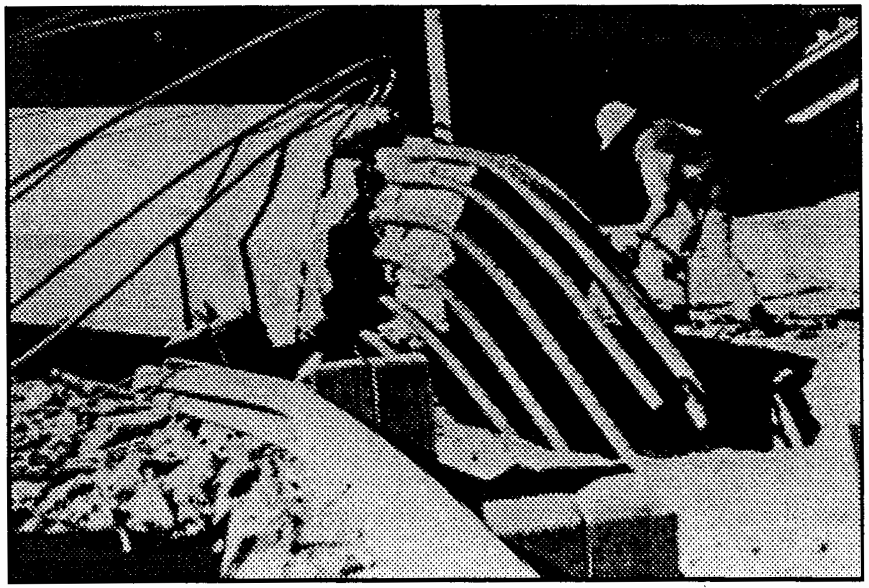
Nature: Limited Ductility
Prevention: Adequate Hinge Zone Confinement

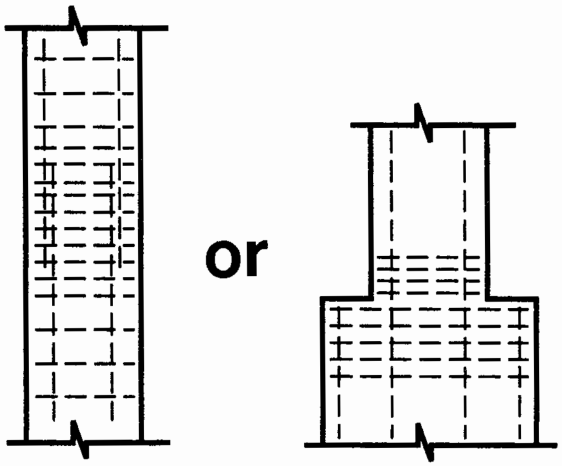




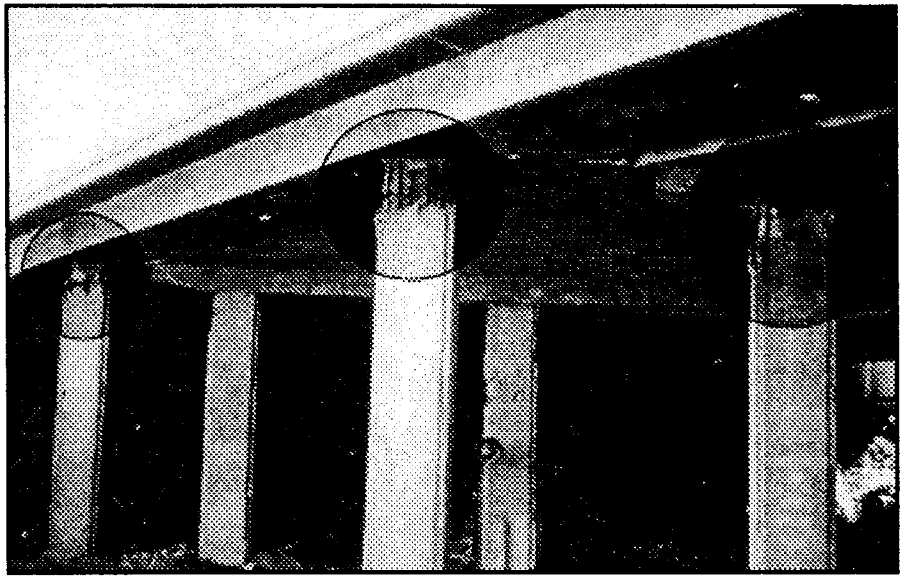
Failure: Insufficient Development

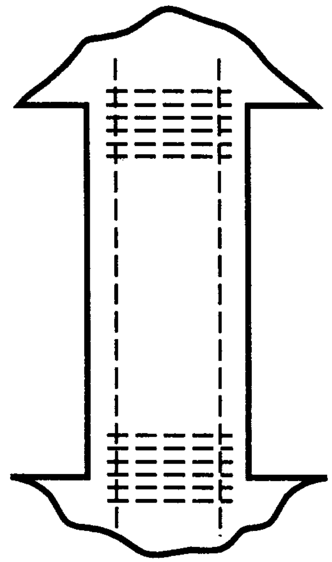
Nature: Limited Ductility
Prevention: Eliminate Splices in High Moment Zones or Confine Splice Heavily





Behavior: Limited Flexural Damage

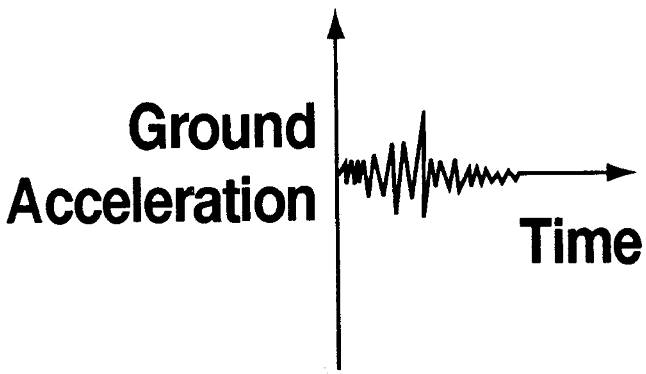


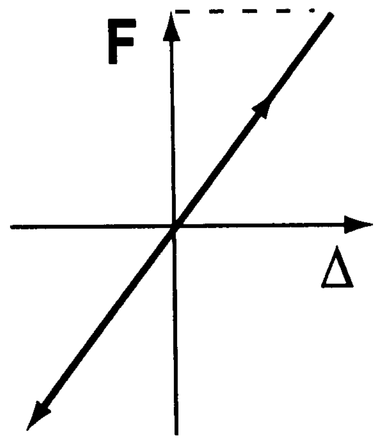


Nature: Ductile

How to Obtain: Sufficient Confinement to Prevent Crushing and Bar Buckling
Also Suppress Shear, Pullout, and Stability Failure

Three Basic Observations





Observation 1

Linear Elastic Systems Develop Large Resisting Forces in Major Earthquakes

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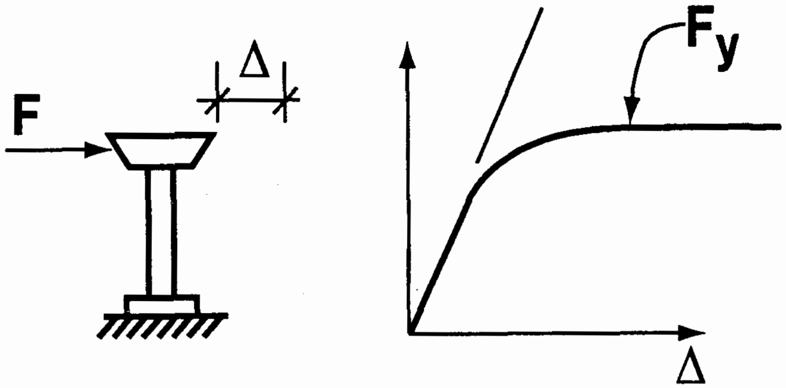
Seismic Bridge Design Applications
25 April 1996, NHI Course Code No. 13063

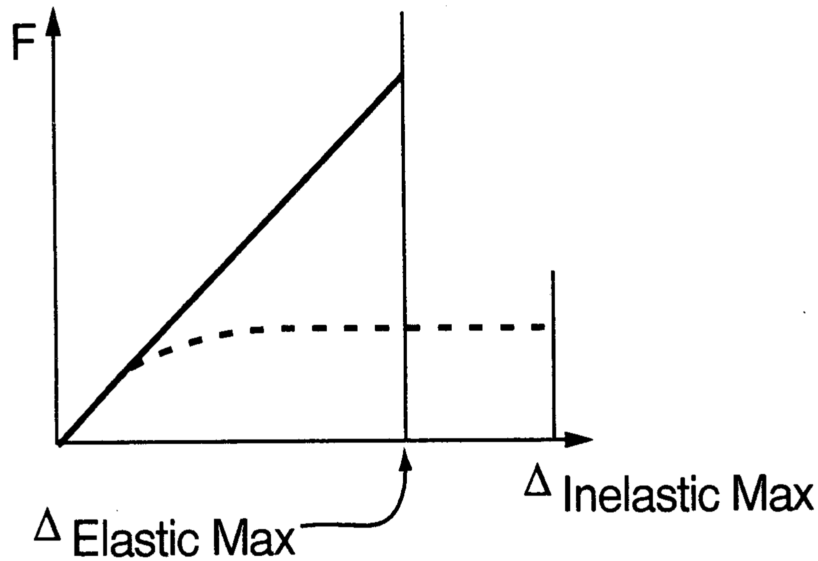
Three Basic Observations (continued)



Observation 2

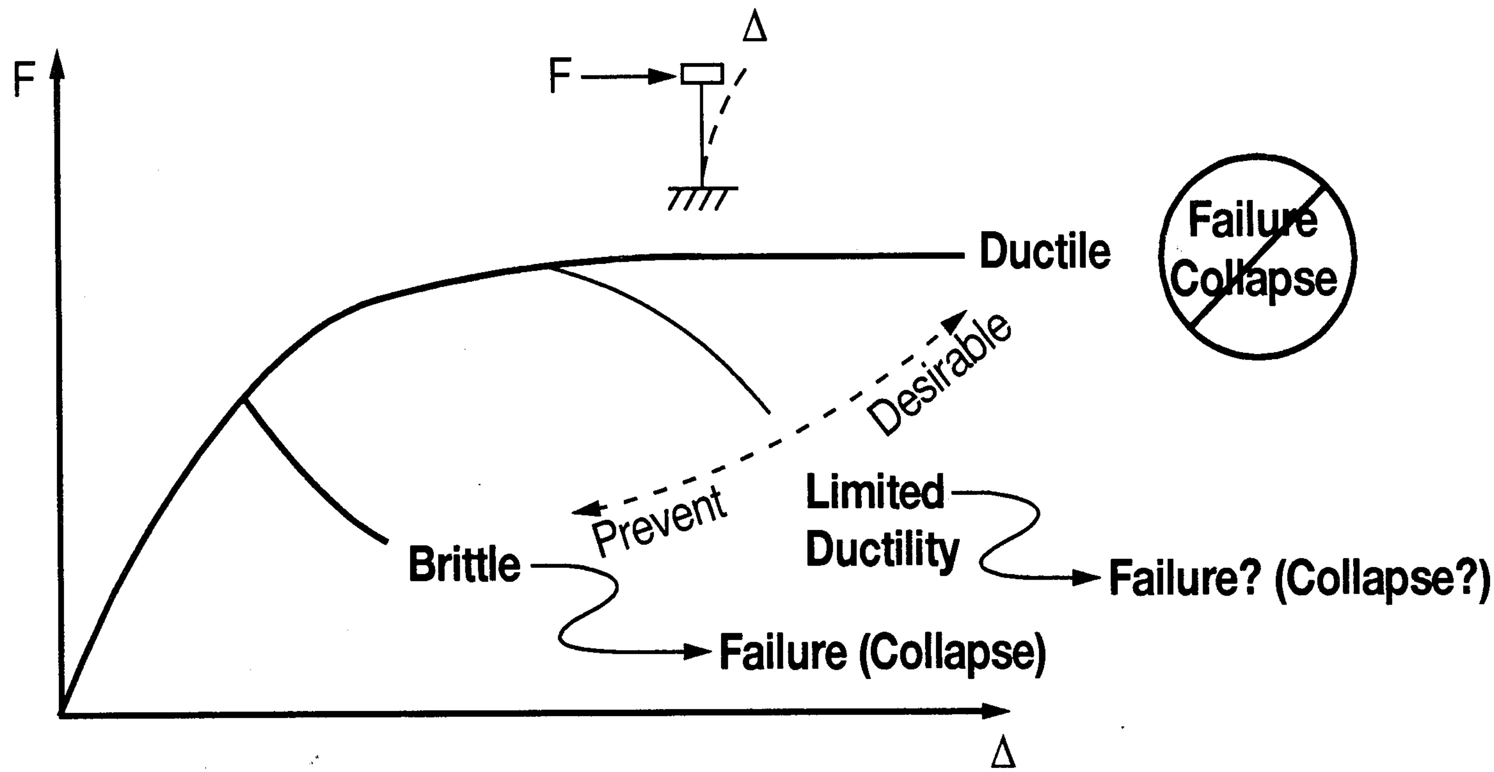
We Can Build Ductile Structures (Ability to Deform into Inelastic Range)

Three Basic Observations (continued)

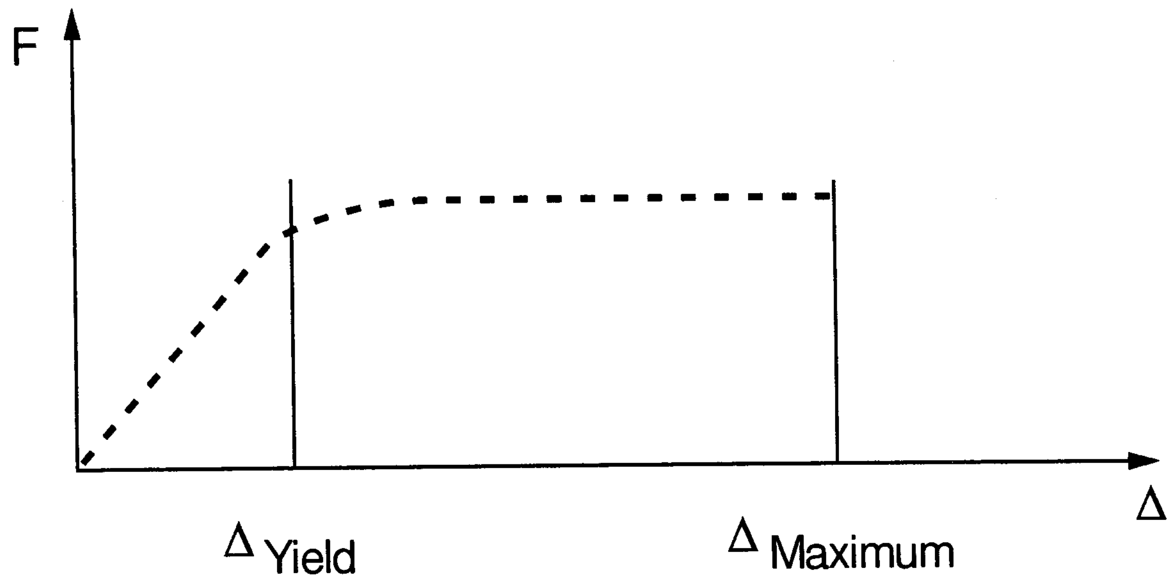


Maximum Displacements of Elastic Systems and Similar Period Yielding Systems Are Roughly Equal

Types of Inelastic Behavior

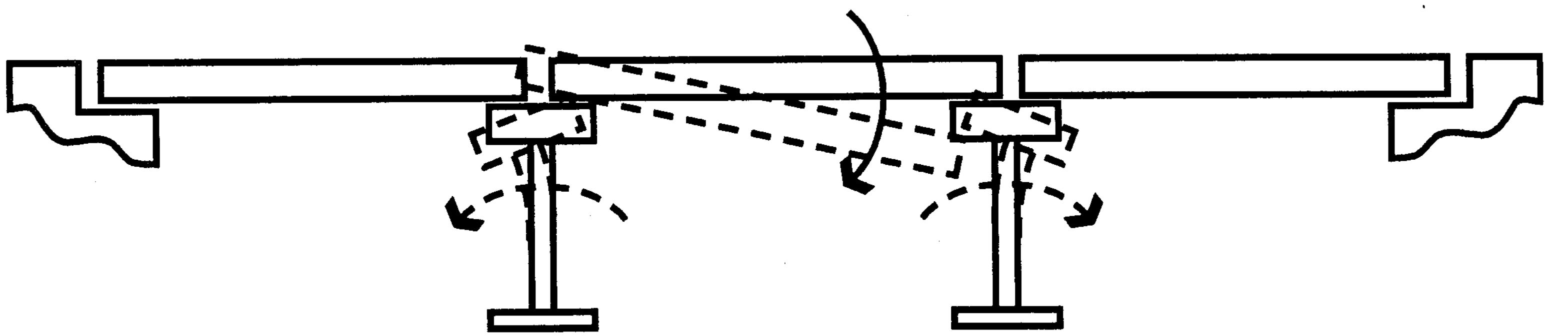


Ductility



$$\text{Ductility } (\mu) = \frac{\Delta_{\text{Maximum}}}{\Delta_{\text{Yield}}}$$

Handling Displacements/Use Conservative Estimates

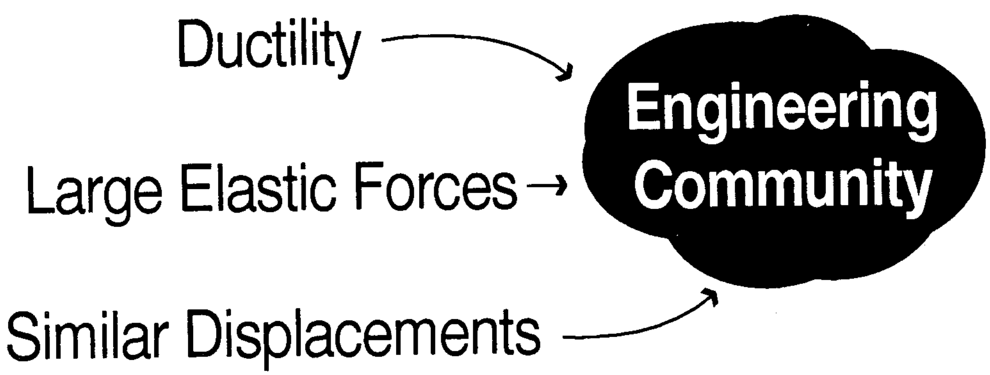


Seat Width Must include Allowance for

- Yielding
- Out-of-Phase Movement of Separate Units

Seismic Design Philosophy

Observations



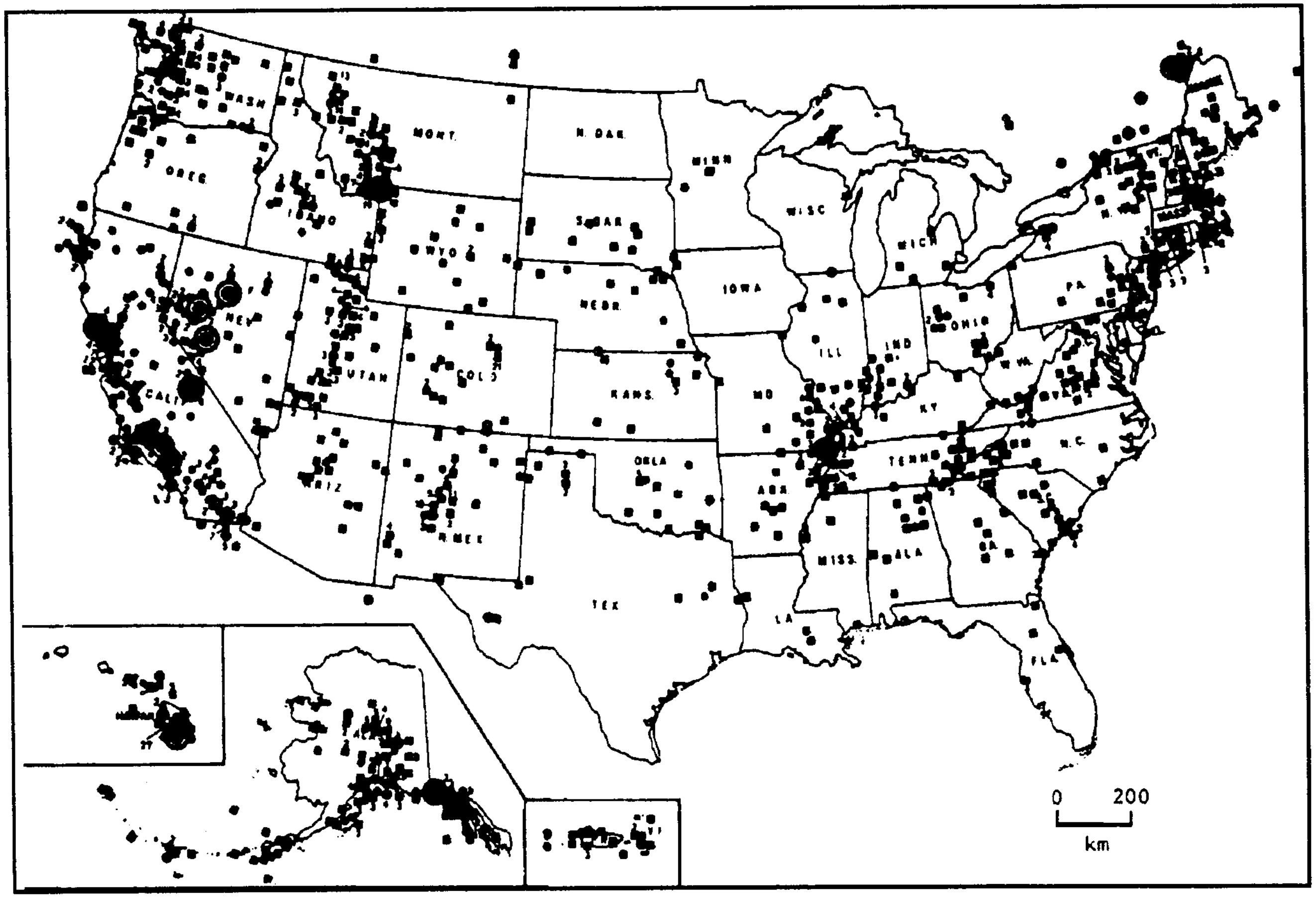
Design Philosophy

- Allow Yielding (Damage) in Major Earthquake
- Damage Should Be Accessible
- No Collapse

Session 1
Seismic Hazard Analysis Concepts

- Regional Importance
- How the Ground Moves
- Where the Seismic Hazard Maps Come From

Earthquake Occurrence in United States

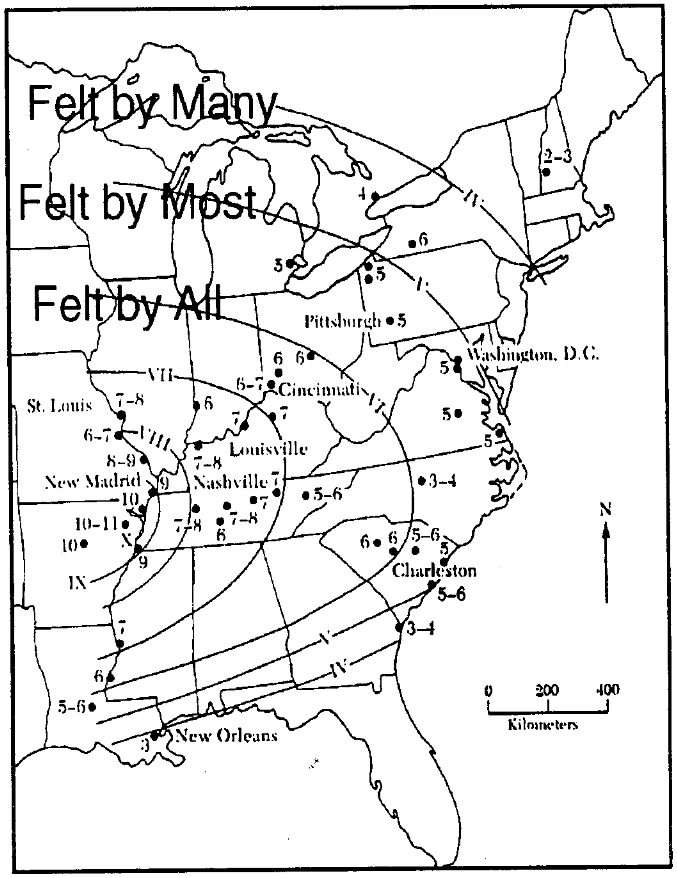


Algermissen

Charleston, South Carolina / 1886

Magnitude = ?? Felt in Boston, Chicago, and St. Louis (All ~ 900 Miles Away)

New Madrid, Missouri / 1811 – 1812



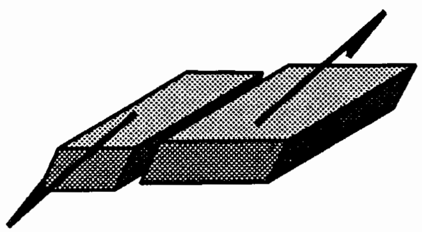
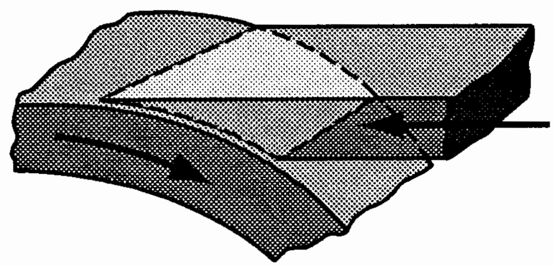
Bolt

3 Main Earthquakes

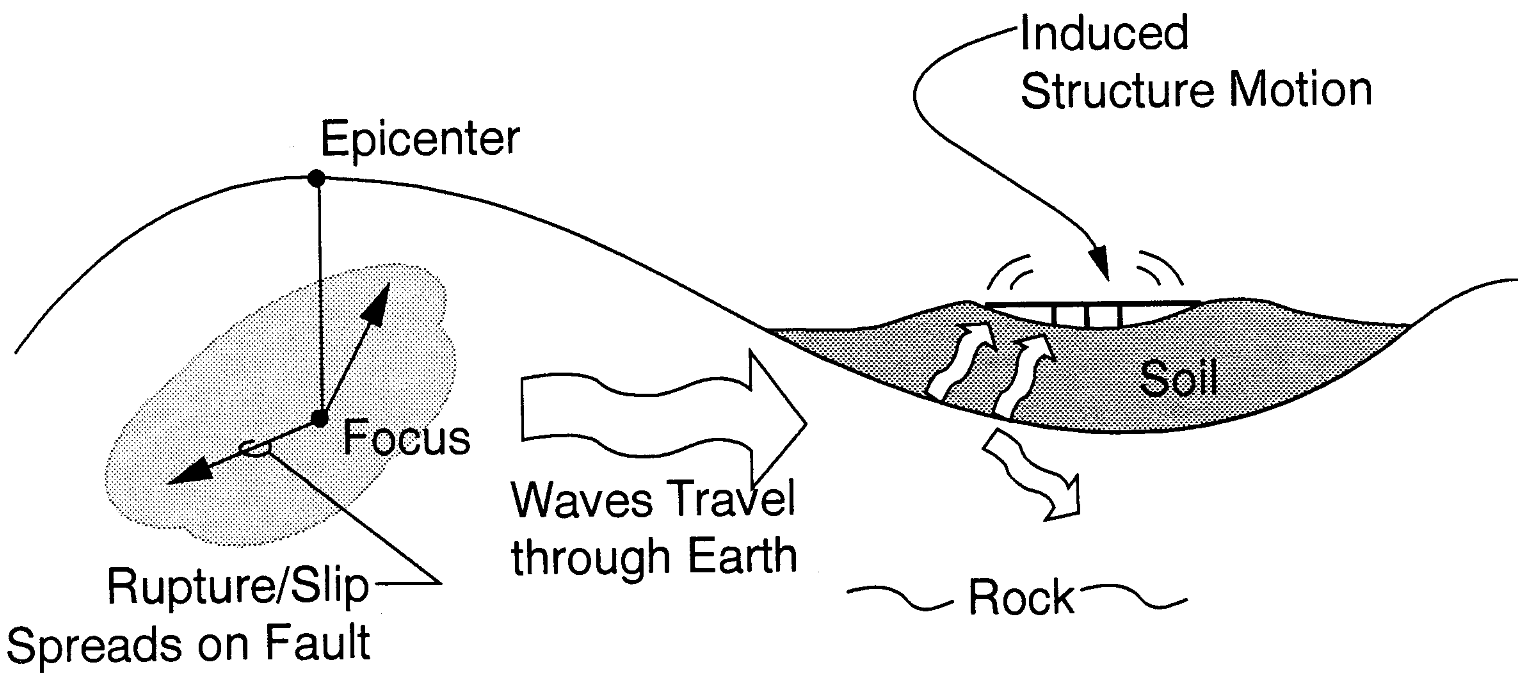
- Magnitudes ~ 7.3 to 7.8
  - December 1811
  - January 1812
  - February 1812
- Chimneys Down in Cincinnati, Ohio
- Falls Formed in Mississippi River

Earthquake Occurrence and Sources

- **Earthquake Occurrence:**
  Primarily at Plate Boundaries — California
  Some Occur within Plates — South Carolina,
  Missouri, etc. — But Not as Often

- **Sources Can Be Identified as:**
  Line (Faults)
  
  Area (Subduction Zones)
  

Earthquake Shaking / Sources-to-Site



Magnitude and Duration Proportional to Area and Amount of Slip

Characterizing Ground Motion for Design

- Use Ground Acceleration
- Need to Go From Earthquake Source to Site Ground Acceleration
- Account for Known Rate of Occurrence

AASHTO I-A:

'Probabilistic' Ground Motion

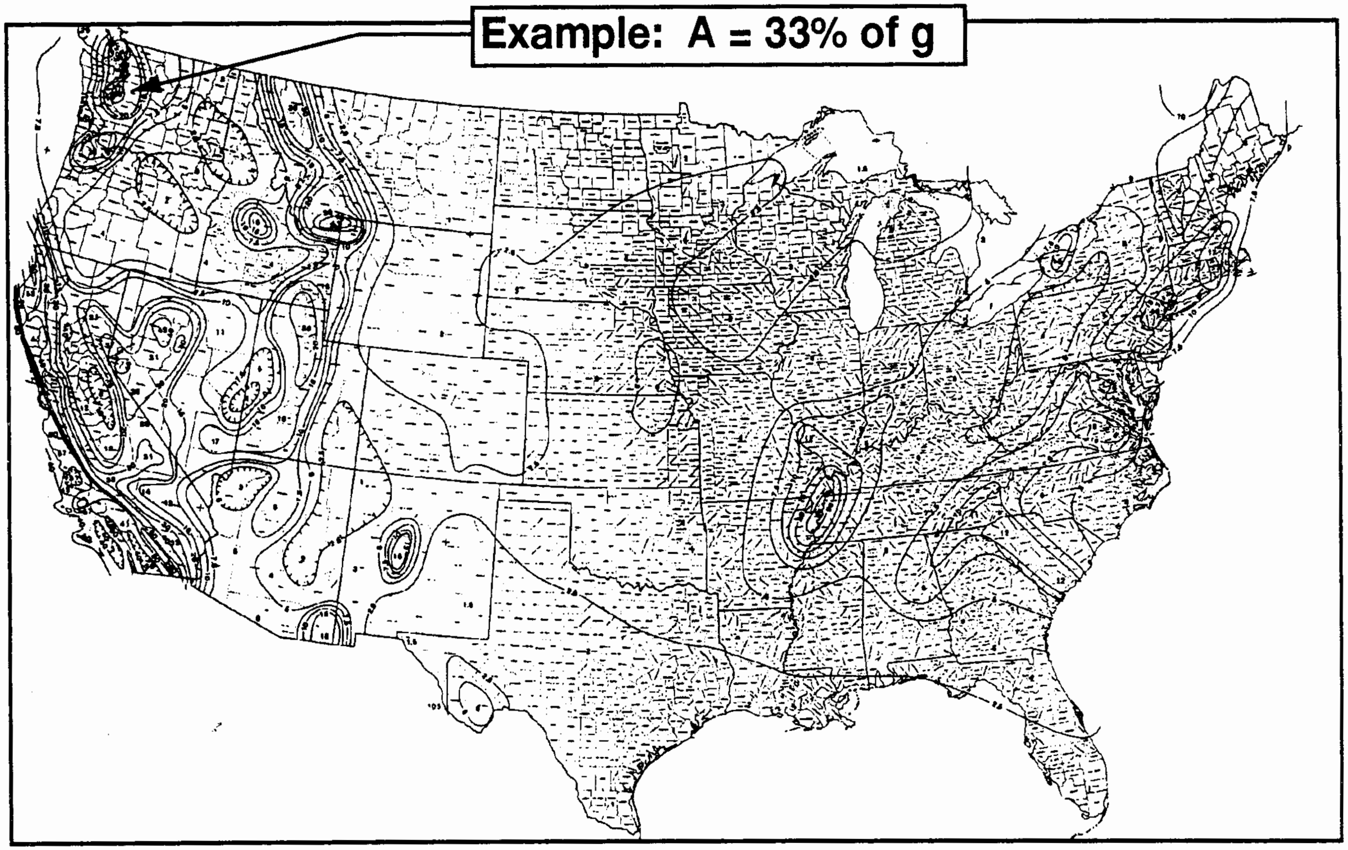
Example:

There Is a 10% Chance that an Earthquake Will Produce an Acceleration that Exceeds 0.33g at a Site During Any 50-Year Interval

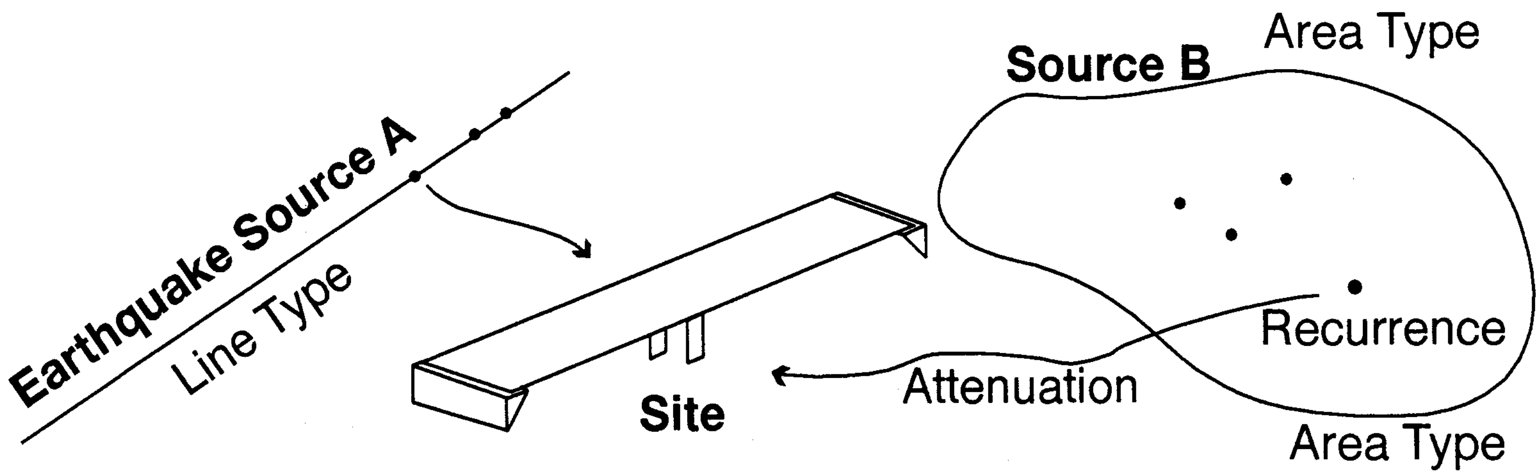
AASHTO 1-A / Acceleration Coefficient, A

A Is Given as Percentage of Gravity

Example: A = 33% of g

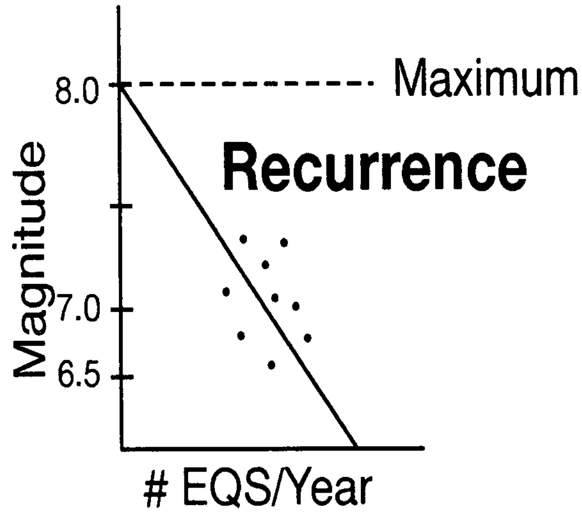


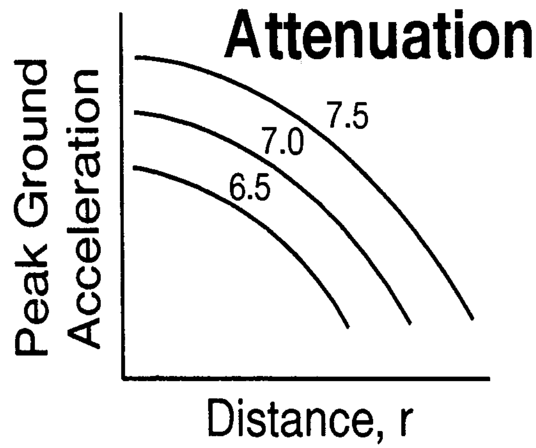
Probabilistic Ground Motion



Probabilistic Ground Motion (continued)

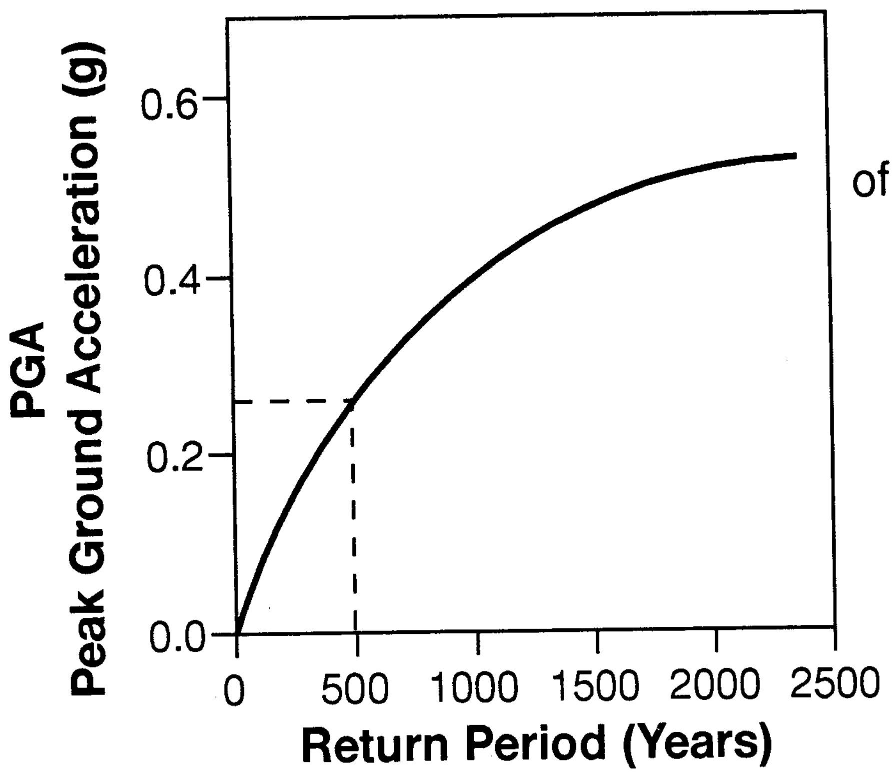
- For Each Source, Know





- For All Sources and All Locations within Source, Add Up Probability that an Earthquake Produces an Acceleration Greater than a Specified Value at the Site for a Given Time Interval

Product of Hazard Analysis

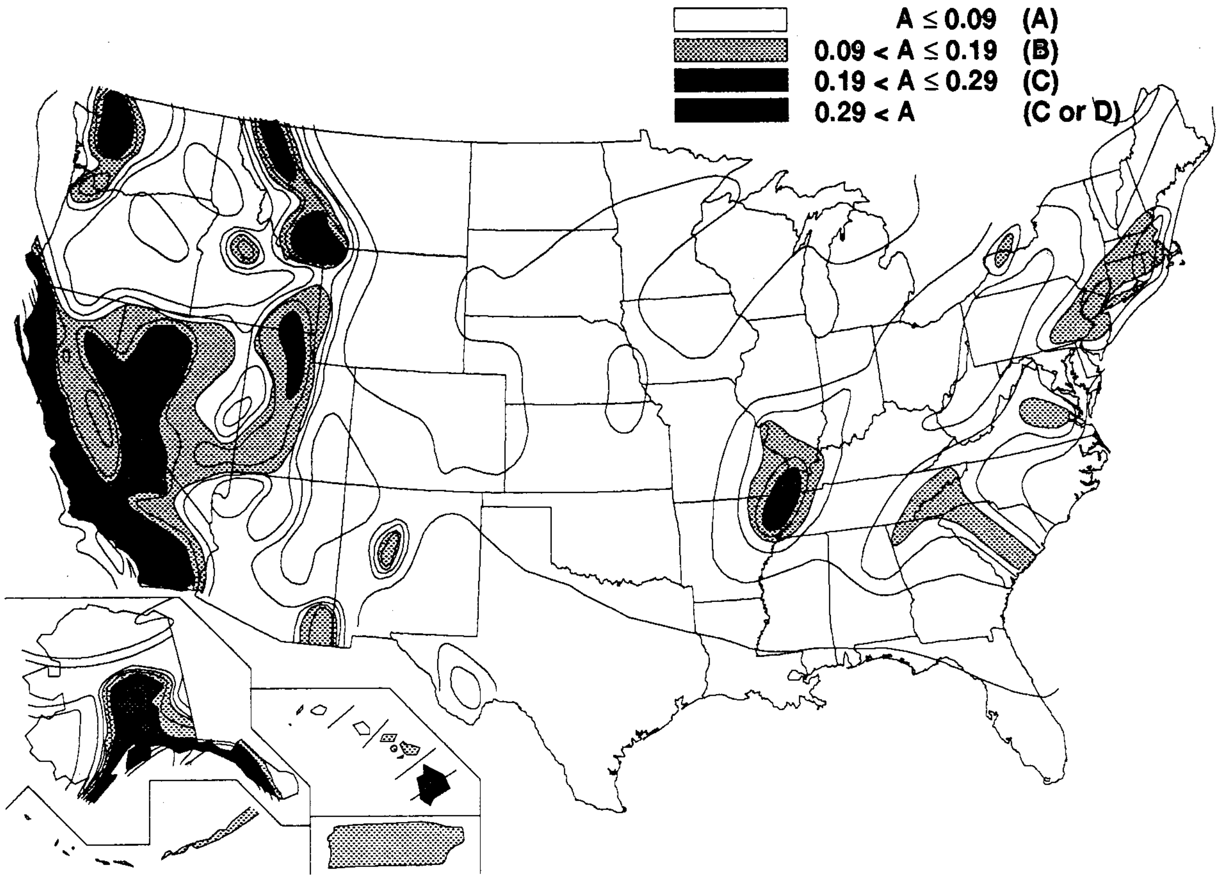


$$\text{Probability of Exceedence} \approx \frac{\text{Time}}{\text{Return Period}}$$

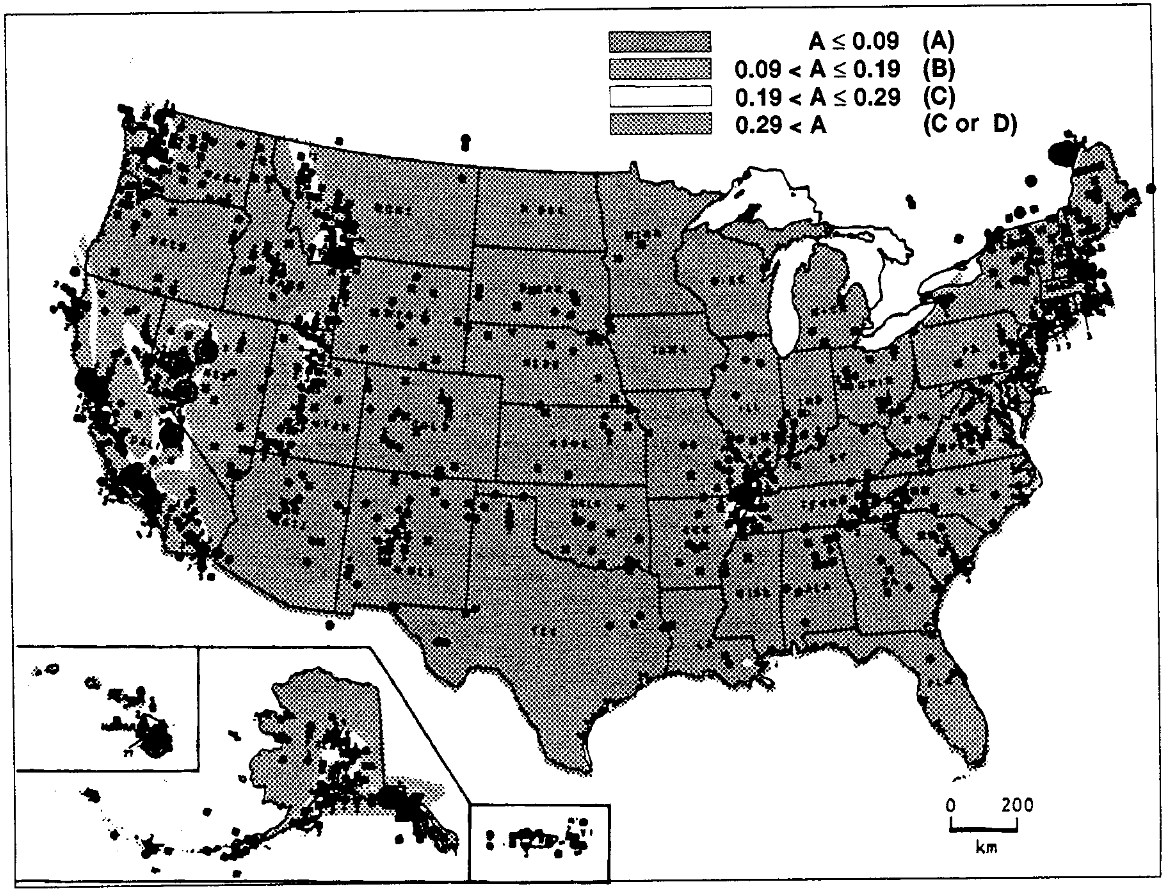
$$0.10 = \frac{50 \text{ Years}}{500 \text{ Years}}$$

(Applies for Low Probabilities)

AASHTO 1-A Acceleration Map



AASHTO Map vs. Occurrences



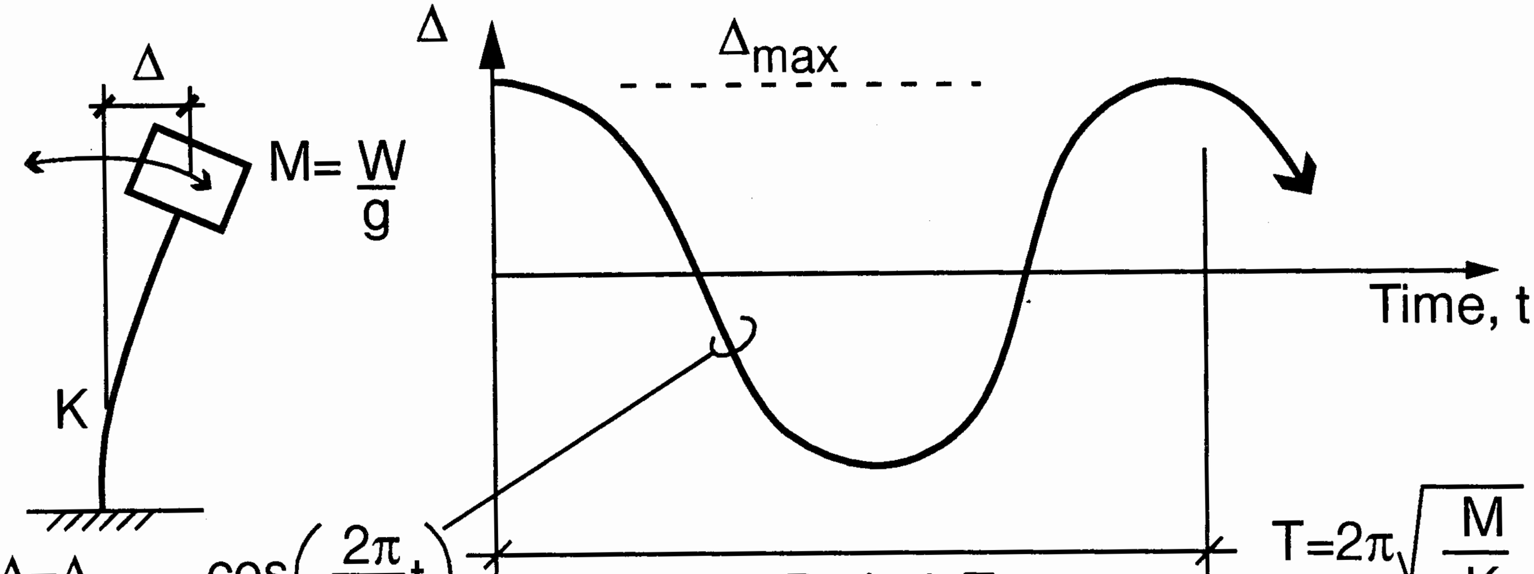
Adapted from Algermissen, 1983, and AASHTO, 1995

Session 2
Structural Dynamics Concepts

- **Single-Mass Systems**
  - Free Vibration
  - Damping
  - Forced Vibration
  - Earthquake Response
- **Multiple-Mass and Distributed-Mass Systems**
- **Response Characterization**

Free Vibration

No Applied Force / Initial Displacement then Release

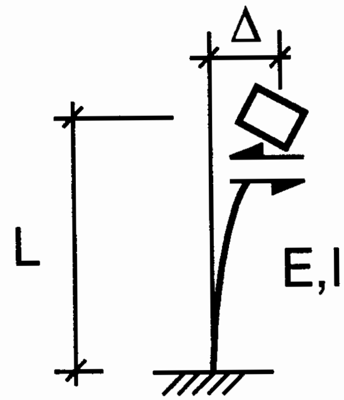


$$\Delta = \Delta_{max} \cos\left(\frac{2\pi}{T} t\right)$$

$$T = 2\pi\sqrt{\frac{M}{K}}$$

Internal Forces / Free Vibration

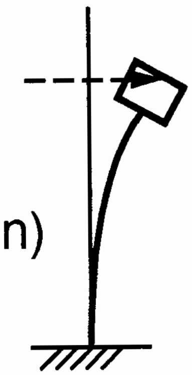
Resisting Force
(Spring Force)



$$f_s = \frac{3EI}{L^3}\Delta$$

Inertial Force

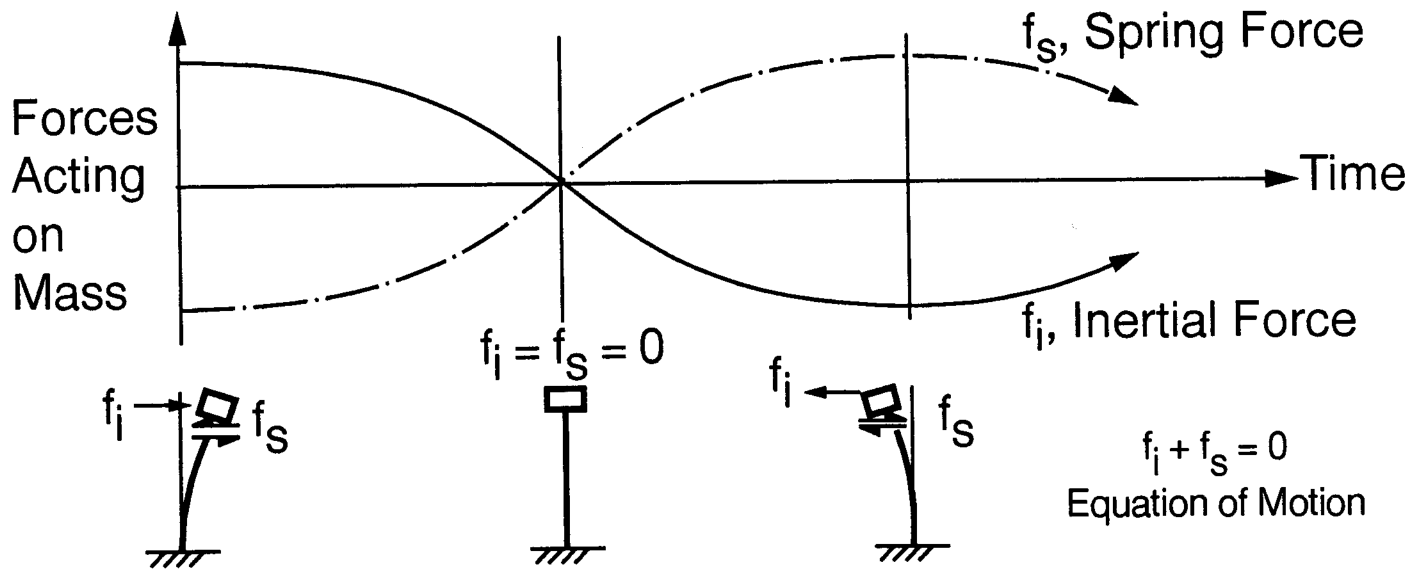
Newton's 2nd Law
(Force = Mass • Acceleration)



$$f_i = ma = m\frac{d^2\Delta}{dt^2}$$

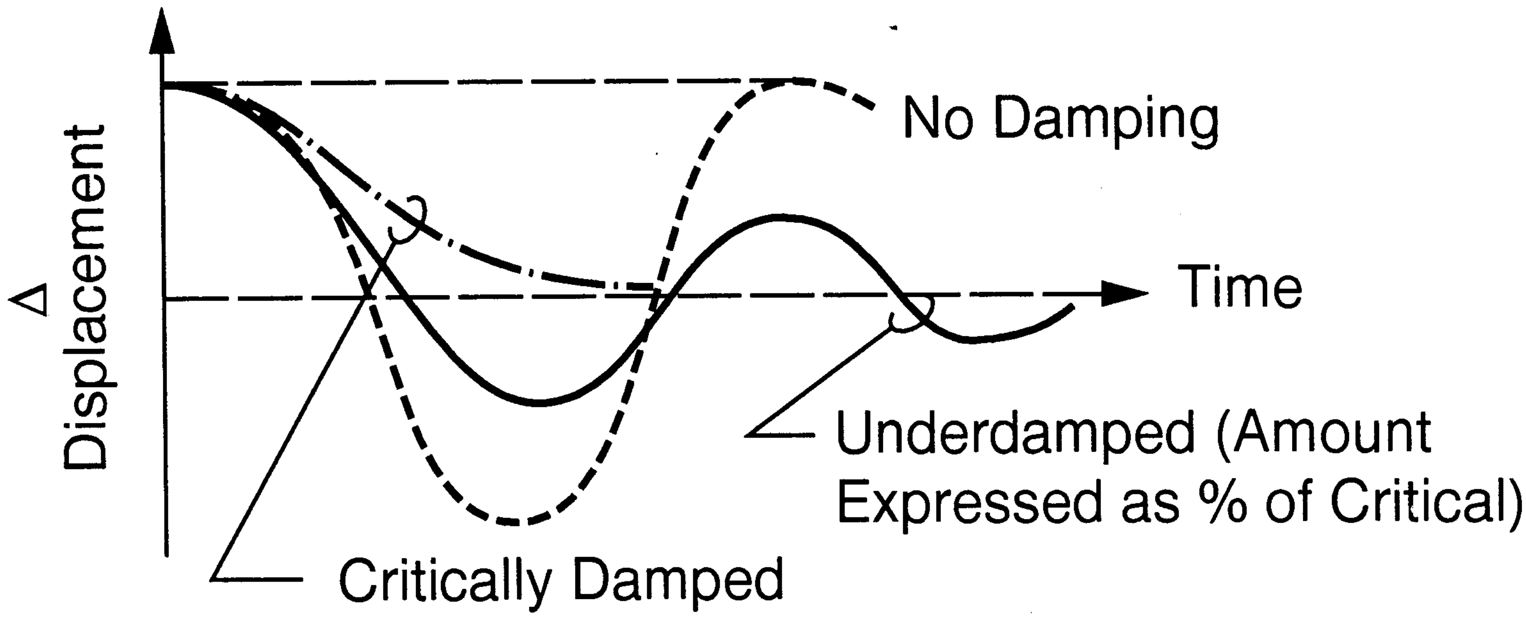
2nd Derivative
of Displacement

Dynamic Equilibrium / Free Vibration



- Structure Vibrates at Period, T; Only 'Vibration Rate' for which Equilibrium Is Satisfied

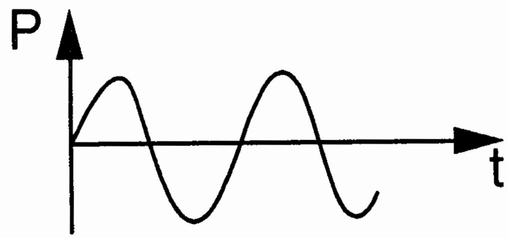
Damping



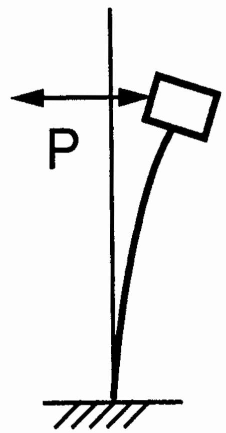
Typical Damping: Value ~ 5%

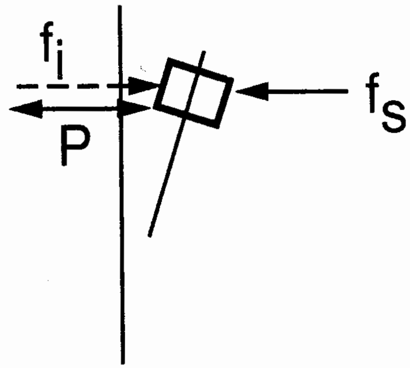
Forced Vibration

Add a Time Varying Load to Our System / No Damping



‘Periodic’
Load





Free-Body Diagram

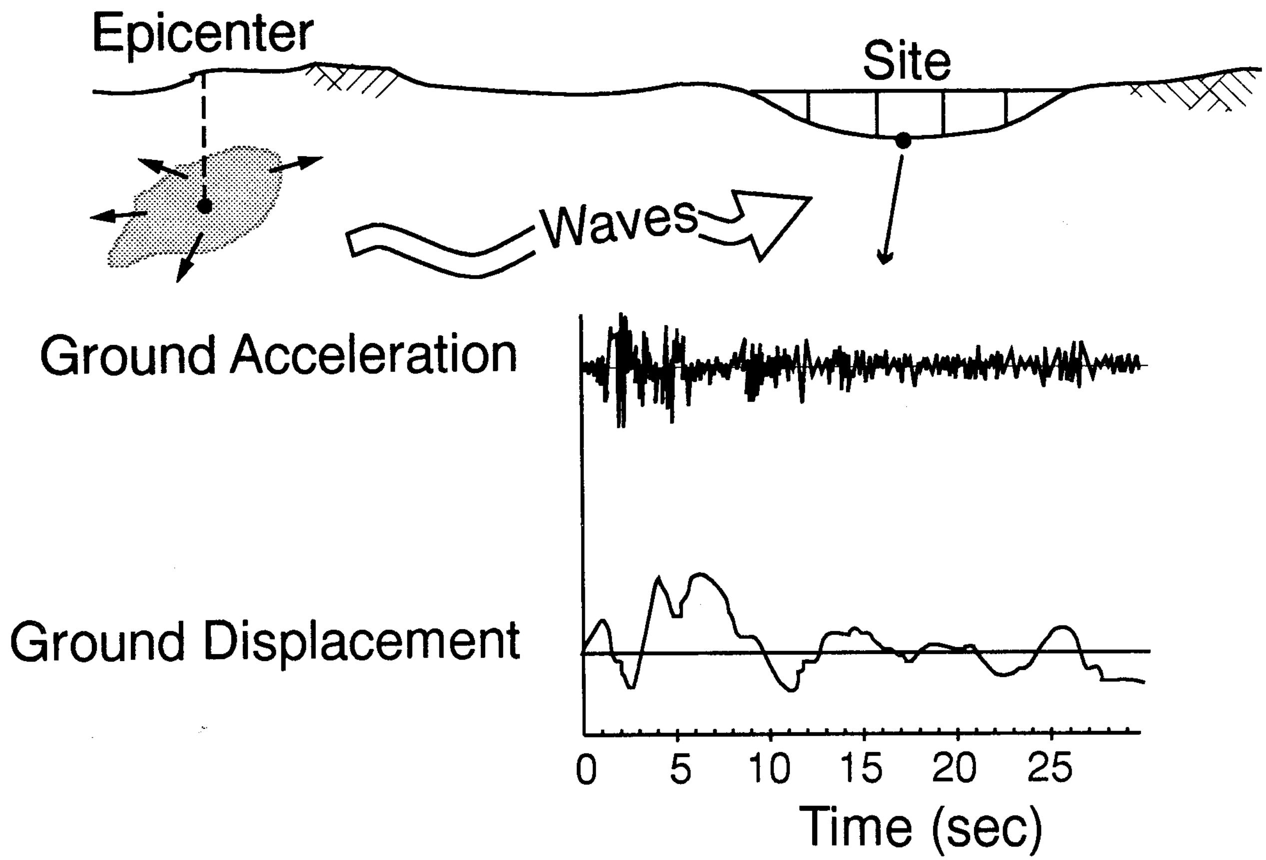
Equation of Motion $\quad P + f_i = f_s$

(Algebraically, the Signs Can Differ)

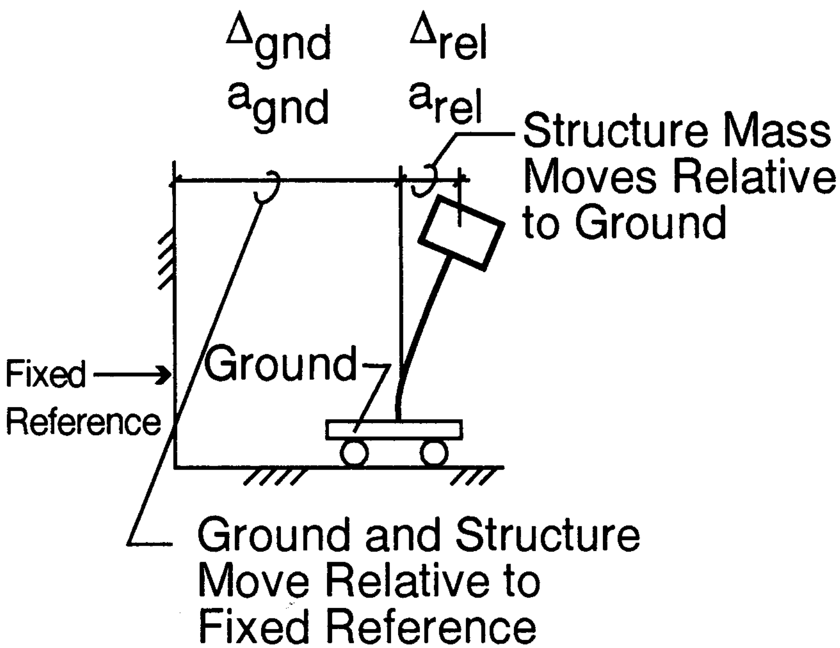
Response Bounds / Periodic Loading

- P Applied 'Slowly' (Relative to Period) – Response Is Essentially Static
- P Applied 'Rapidly' – Response, Δ, Is Small
- Intermediate Case – Response Can Be Large
  'Amplification' $\longrightarrow$ Resonance
  (As Loading Period Approaches Structure Period)

Earthquake Effects



Earthquake 'Loading' – Snapshot



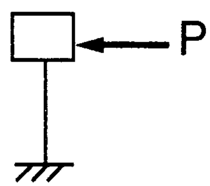
Inertial Force:

$$f_i = \underbrace{m(a_{gnd} + a_{rel})}_{\text{Total Acceleration of Mass}}$$

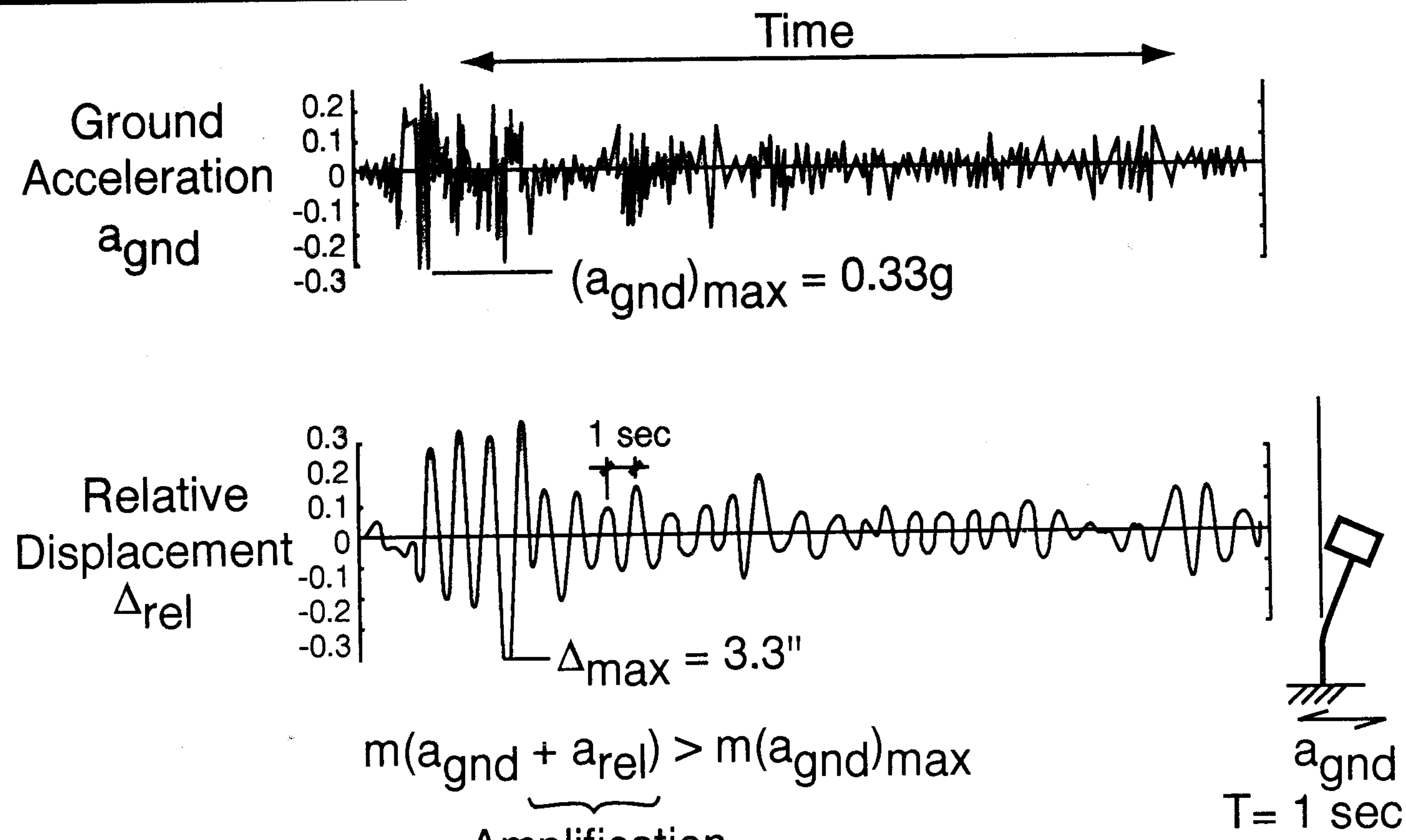
Equilibrium:

$$f_i = f_s$$

$$\underbrace{ma_{gnd}}_{\text{Equivalent to a 'Load', P}} + ma_{rel} = k\Delta_{rel}$$

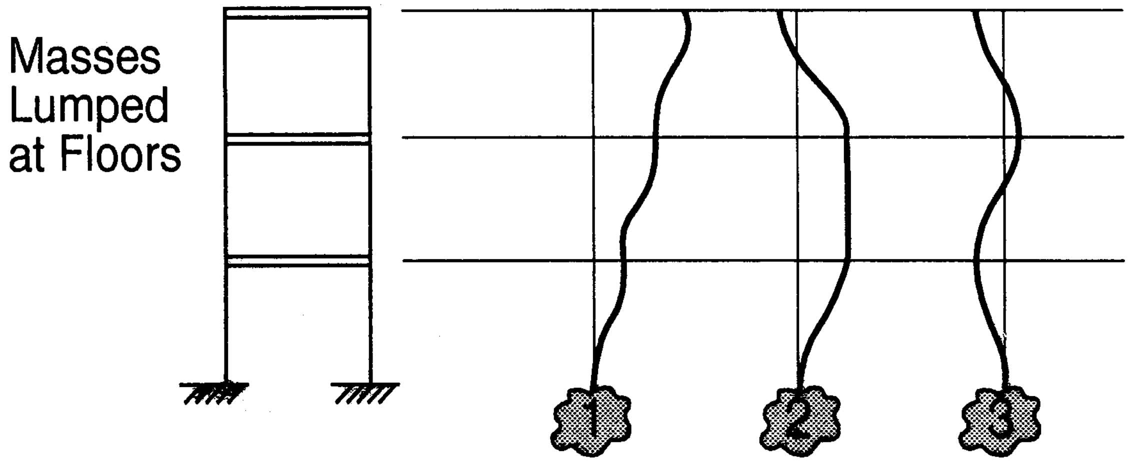


Earthquake Response



$$m(\underbrace{a_{gnd} + a_{rel}}_{\text{Amplification}}) > m(a_{gnd})_{max}$$

Response with Distributed Mass Or Multiple Masses



- More than One Mass Leads to More Vibration 'Modes' and 'Periods'

- Modal Periods, T_i = Function of Shape i and $T_1 > T_2 > T_3$ etc.

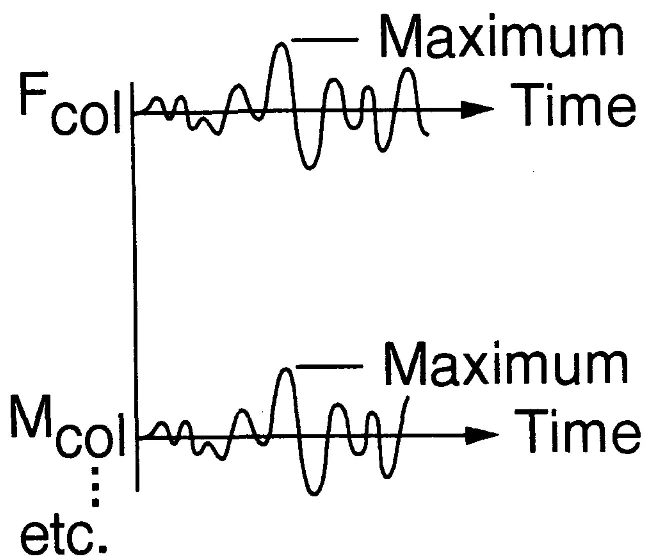
Multi-Modal Response Basics

- Number of Modes Set by Number of Masses and Their Freedom to Move (Dynamic Degrees-of-Freedom, n)
- Equilibrium Satisfies n Simultaneous Equations yuk!
- Use Computer, but Understand Conceptually!
- Response (Linear Elastic) Is Superposition of n Modal Responses
  Forces, F
  Displacements, Δ
- Not All Modes Are Required to **Estimate** Response

How to Characterize Response

- **Complex Method**

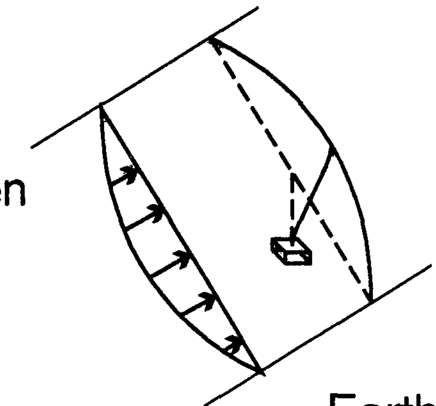
  Full Time History



How to Characterize Response (continued)

- **Simple Method**

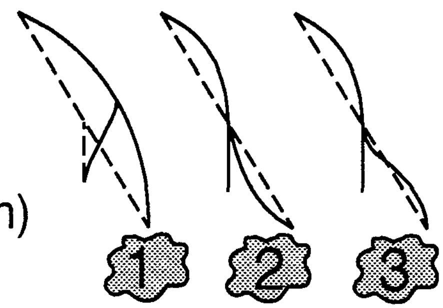
  Quasi-Static (**Maximum** of One Approximate Mode Often Sufficient)



Earthquake Load
(including Amplification)

- **Intermediate Method**

  Multimode Superposition
  (Find Maximum of n Actual Modes, then **Combine** to **Estimate** Actual Maximum)



Session 2
Response Spectra

- Definition
- Amplification and Period
- Effects of Site Soil Condition
- AASHTO Design Spectra
- How to Use Spectra

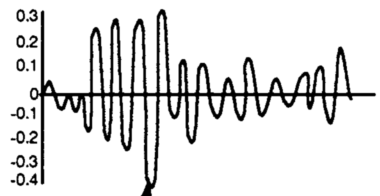
From Earthquake to Design

Single Earthquake

a_{gnd}

↓

a_{total}



↓

$max(a_{total})$ max

↓

$max(force) = mass \bullet max(a_{total})$

For Design

$max(a_{gnd}) = ?$ A — from Map

↓

Typical Amplification Factor = ?

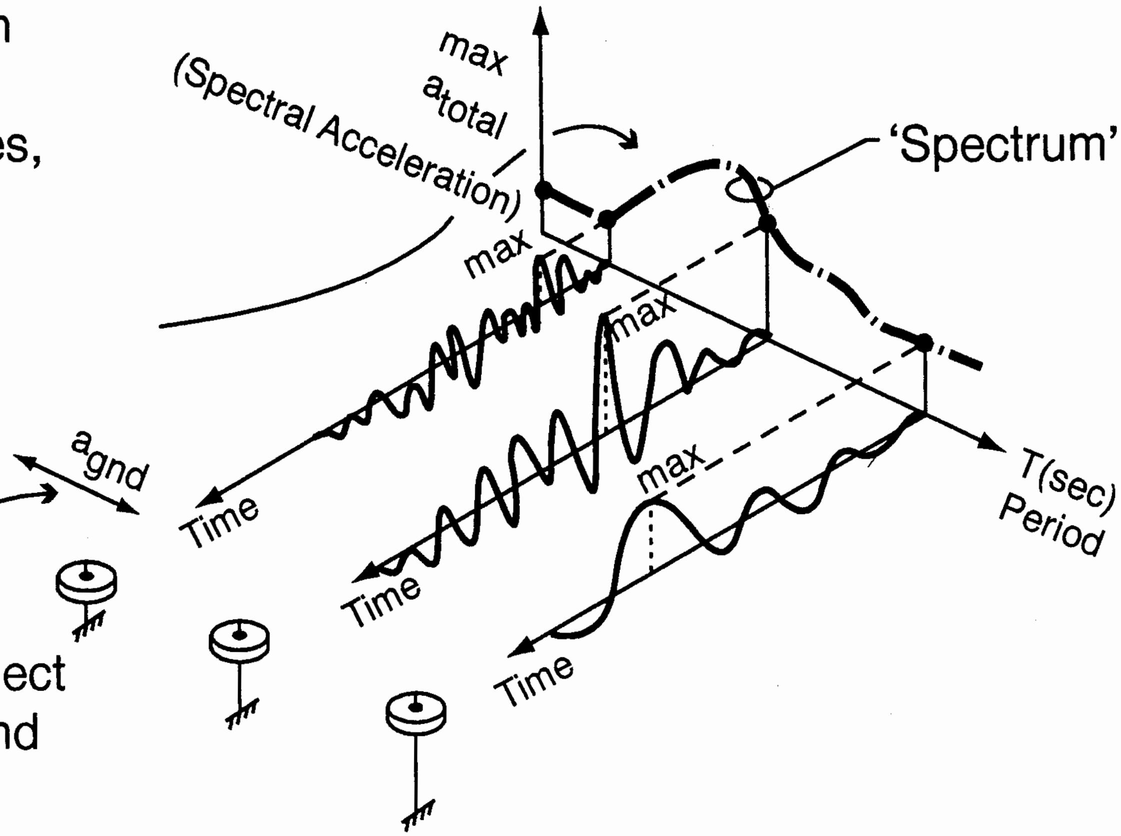
$$\frac{max\ (a_{total})}{max\ (agnd)} = \Omega$$

↓

$max(force) = mass \bullet A \bullet \text{'}\Omega\text{'}$

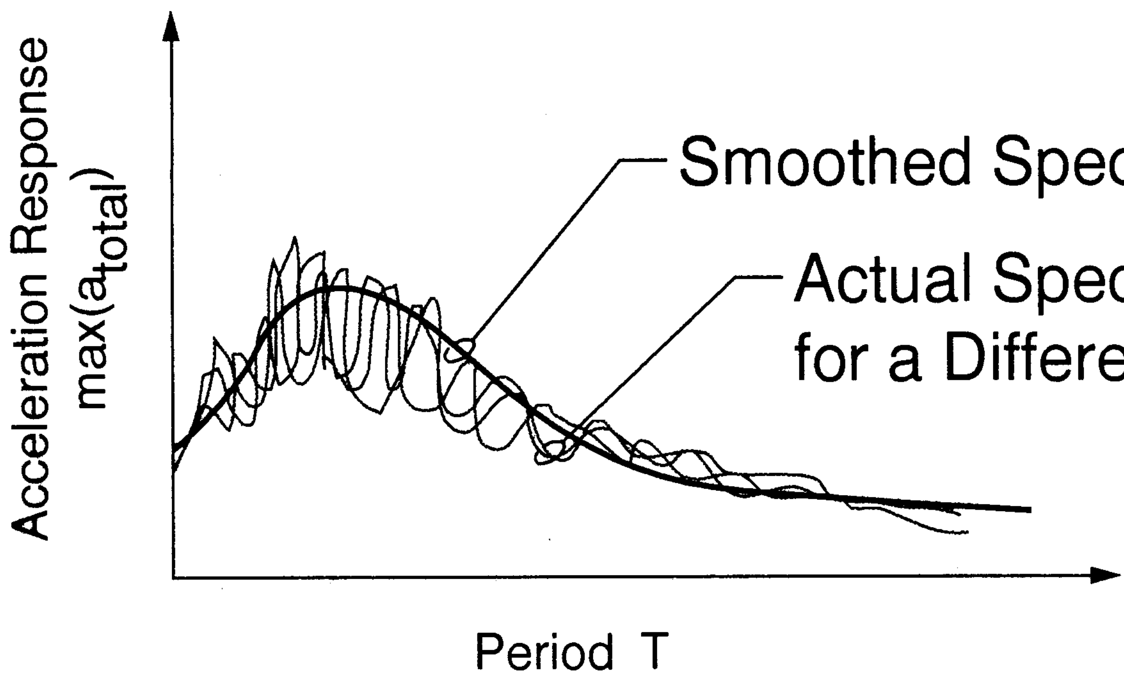
Define Response Spectrum

Determine Maximum Response for a Group of Structures, All with Different Periods; then Plot

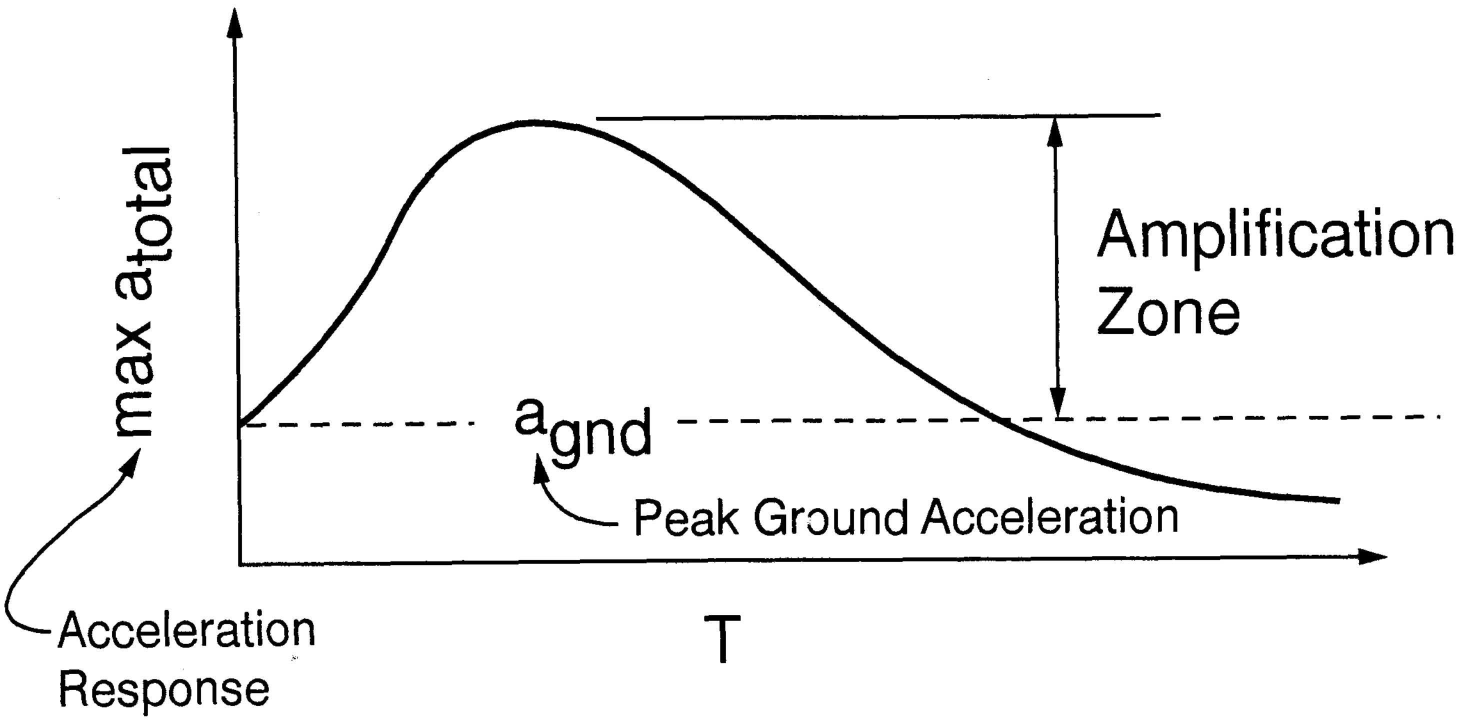


All Structures Subject to the Same Ground Motion

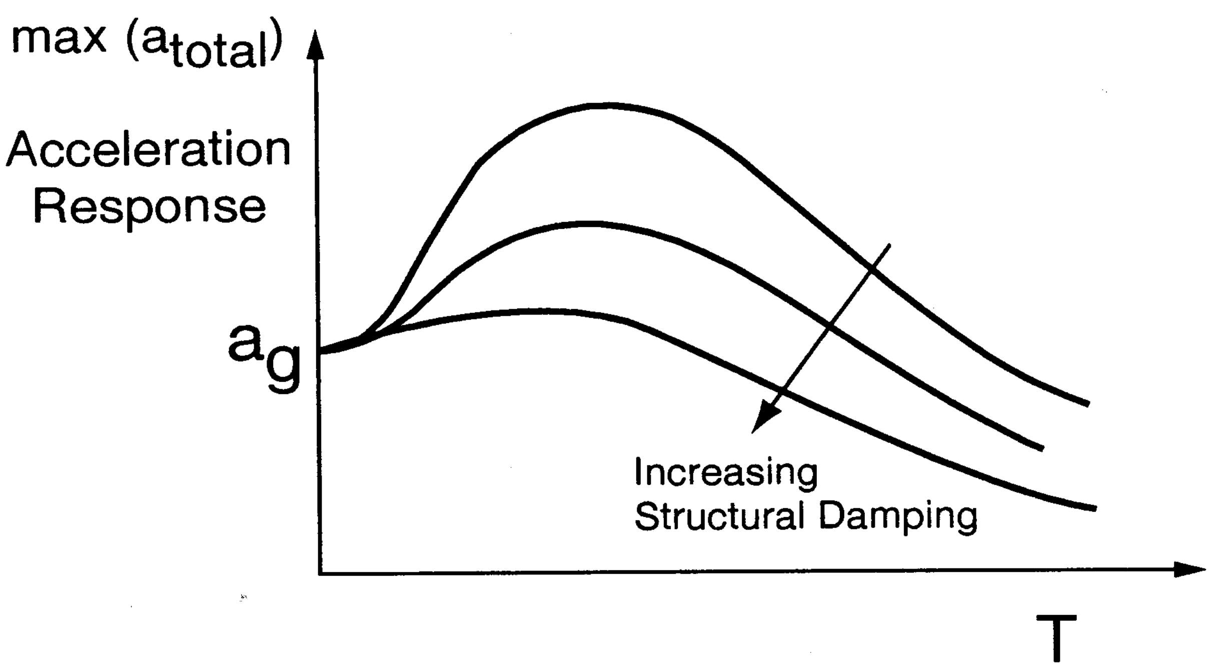
Actual Spectra vs. Smoothed Spectrum



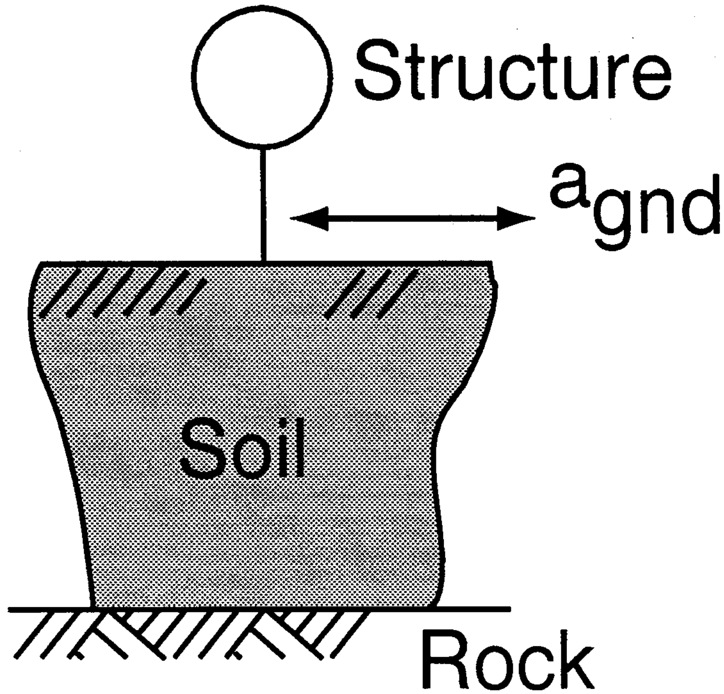
General Shape of Spectra



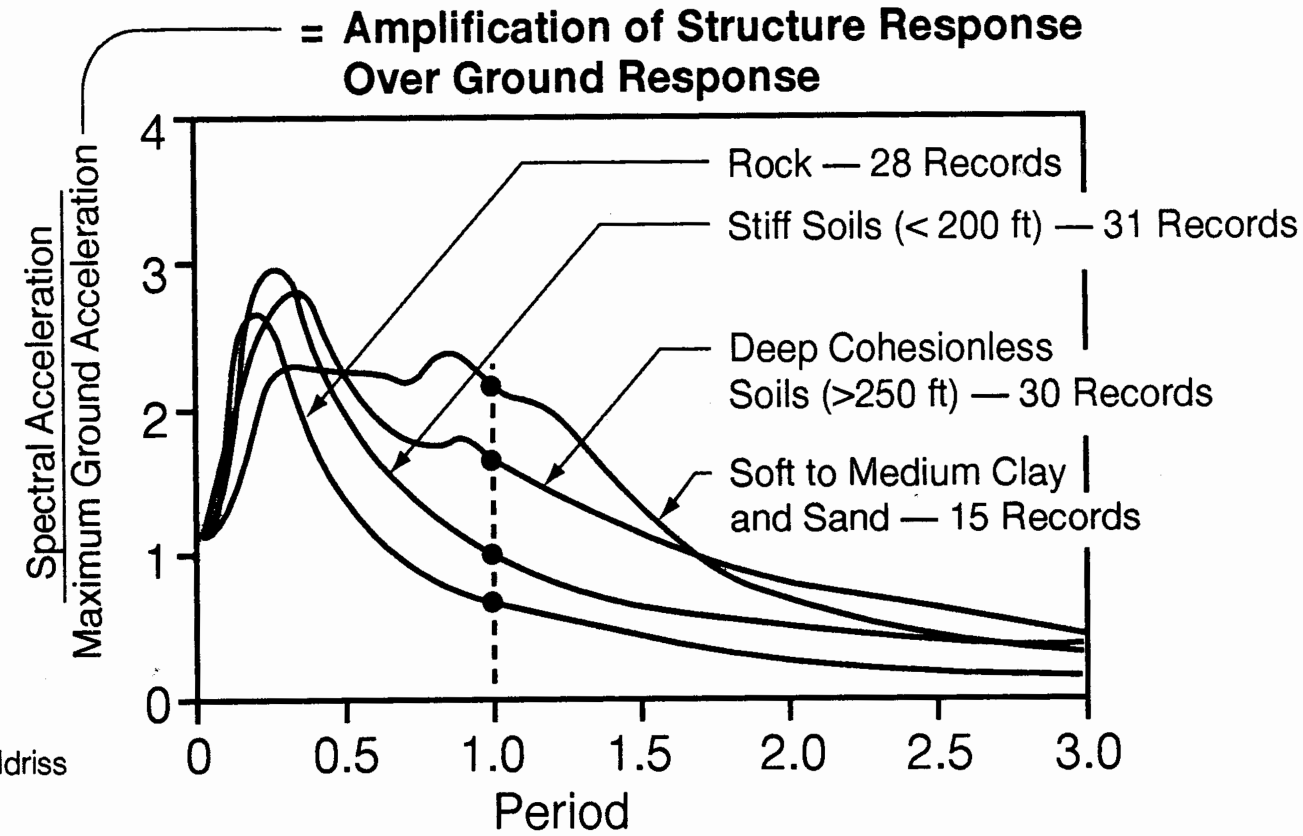
Effects of Damping



Effects of Site Soil Conditions

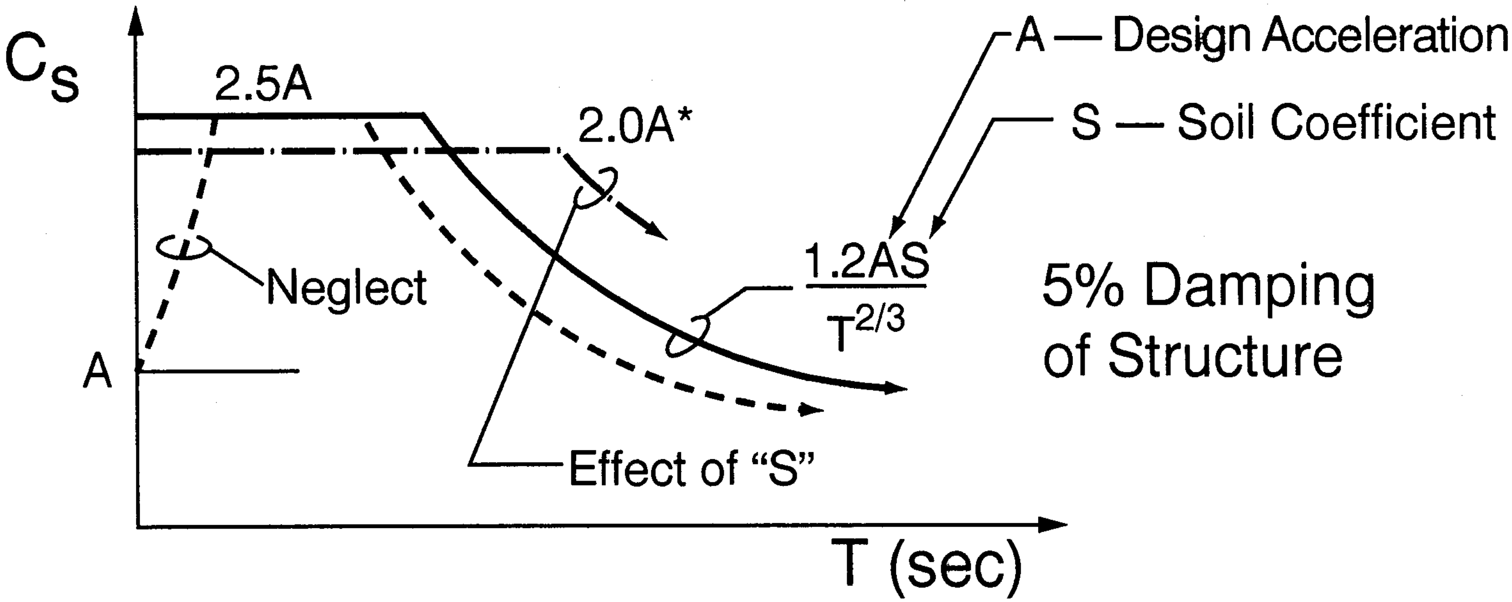


Effects of Site Soil Conditions



Seed & Idriss

AASHTO Design Spectra



5% Damping of Structure

Total Acceleration or 'Spectral Acceleration'

$a_{total} = C_S \cdot g$ ← Acceleration Due to Gravity

* 2.0A – C_S Cap for Soft Soil when $A \geq 0.30$

Site Coefficient, S

Table 2. Site Coefficient (S)

	Soil Profile Type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

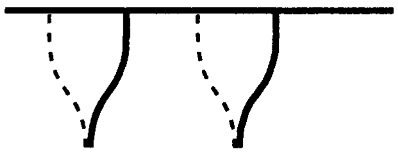
I. Rock or Stiff Soil < 200 ft Thick

II. Deep Stiff Soil > 200 ft Thick

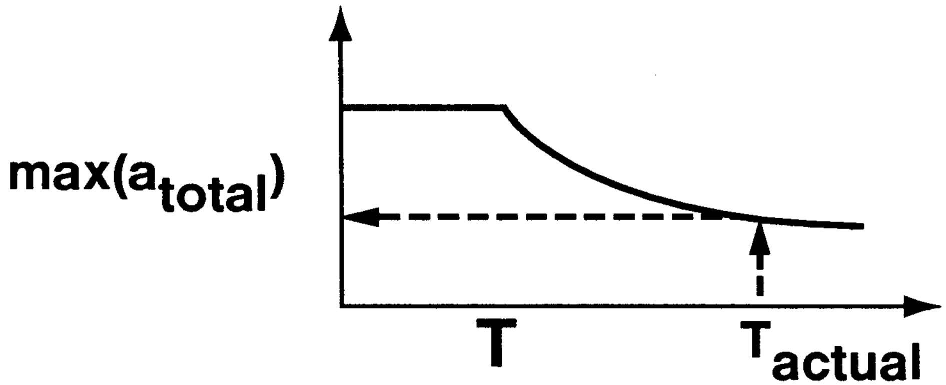
III. Soft to Medium Clays and Sands > 30 ft Thick

IV. Soft Clay or Silt > 40 ft Thick

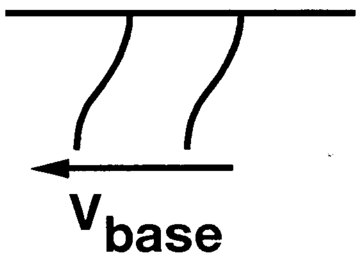
How to Use a Response Spectrum



Determine Period of Bridge, T



Use Spectrum to Find $max(a_{total})$



$$V_{base} = \text{Weighting Factor} \bullet \frac{W}{g} \bullet C_s$$

Session 2
Overview of AASHTO Division 1-A

- Seismic Performance Category
- Choosing an Analysis Technique
- Response Modification Factors
- Overall Flow

Seismic Performance Categories, SPC

Seismic Hazard, A from Map

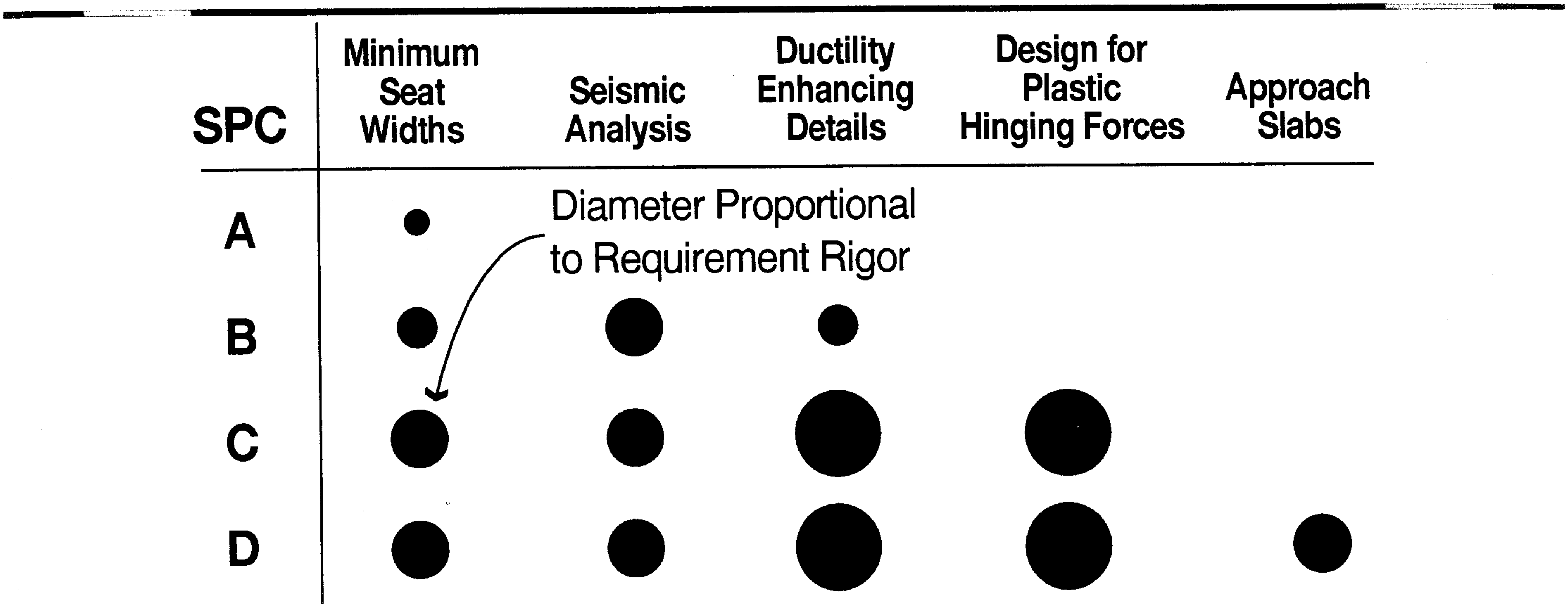
Bridges	IC
Essential	I
Other	II

TABLE 1. Seismic Performance Category (SPC)

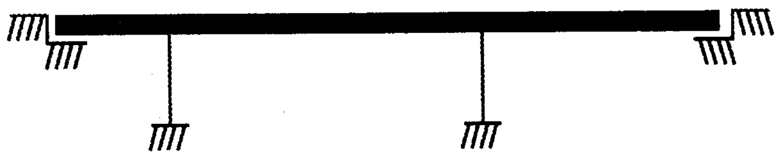
Acceleration Coefficient	Importance Classification (IC)	
A	I	II
$A \leq 0.09$	**A**	**A**
$0.09 < A \leq 0.19$	**B**	**B**
$0.19 < A \leq 0.29$	**C**	**C**
$0.29 < A$	**D**	**C**

SPC

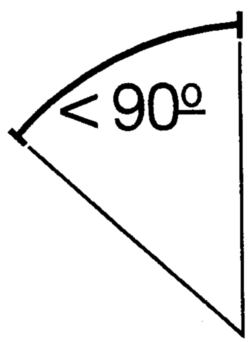
Design Requirements Tighten As Category Increases



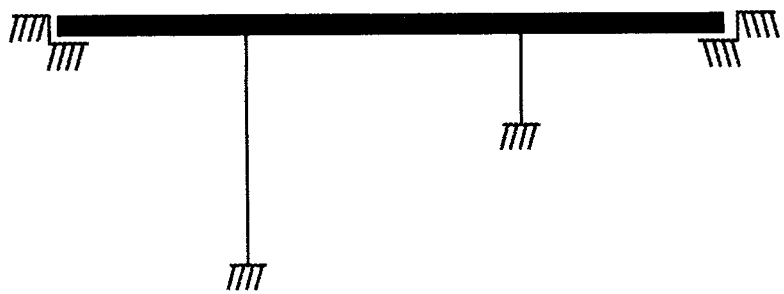
Regular Bridges / 2 to 6 Spans



Span Length Ratio



Curvature in Plan



Pier Stiffness Ratio

Minimum Analysis Requirements

TABLE 4. Minimum Analysis Requirements

Seismic Performance Category	Regular Bridges with 2 Through 6 Spans	Not Regular Bridges with 2 or More Spans
A	Not required	Not required
B, C, D	Use procedure 1 or 2	Use procedure 3

Seismic Analysis Procedures

1. Uniform Load Method
2. Single-Mode Spectral Method
3. Multimode Spectral Method
4. Time History Method

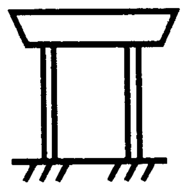
Increasing Complexity

Flow of Analysis Procedures

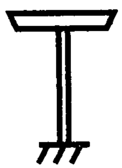
Stiffness and Weight	Period	Lateral Force Coefficient	Base Shear	Forces and Displacements
$\left.\begin{matrix}K\\W\end{matrix}\right\}$ →	T →	C_s →	V →	$F_{internal}$
	$2\pi\sqrt{\dfrac{W}{gK}}$	$\dfrac{1.2AS}{T^{2/3}}$	C_sW	$\Delta_{structure}$

Response Modification Factor, R

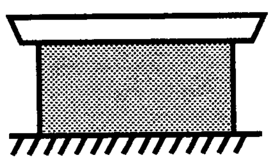
$$M_{design} = M_{elastic}/R$$



High Redundancy, Ductile — $R = 5$

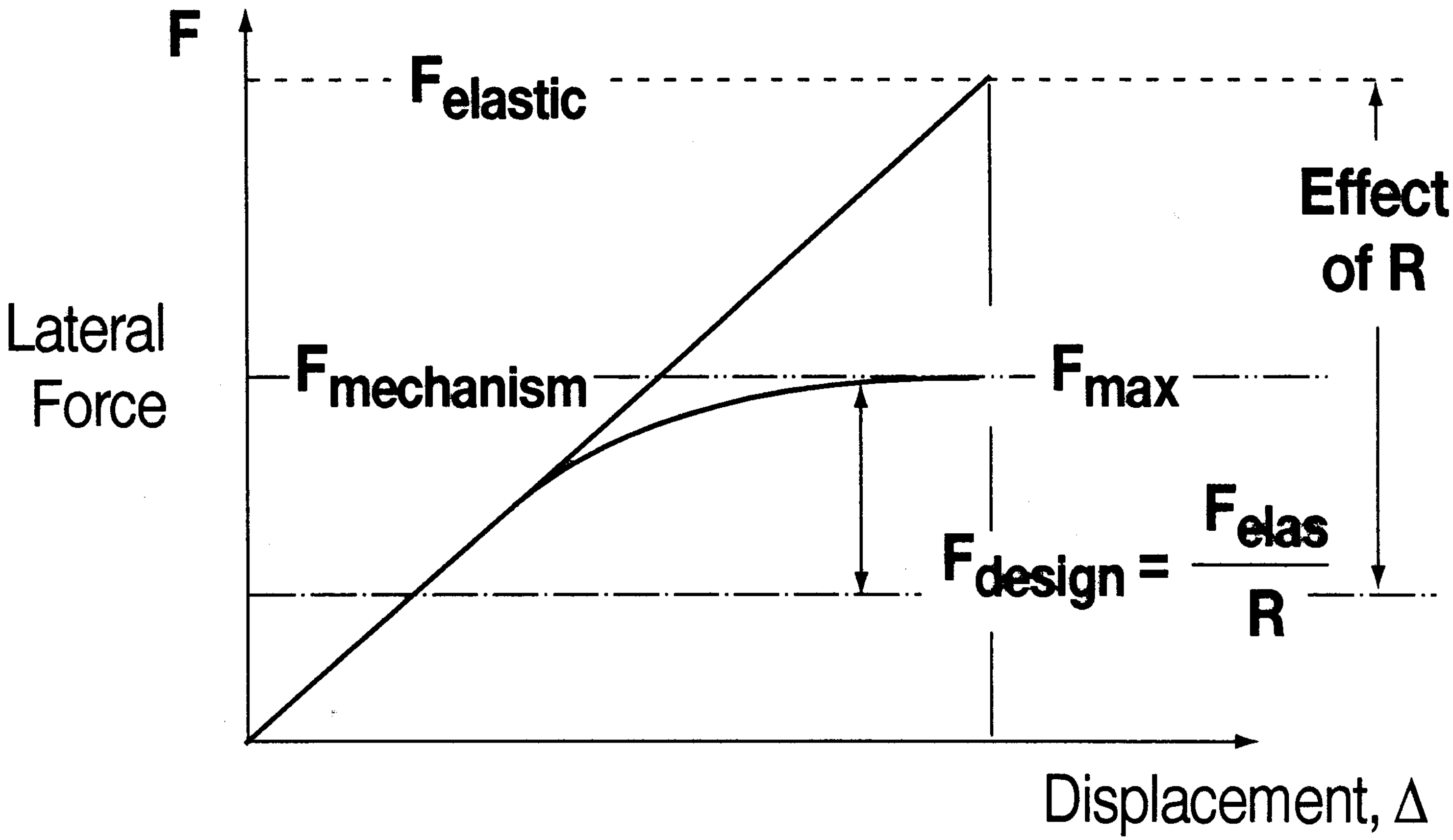


No Redundancy, Ductile — $R = 3$

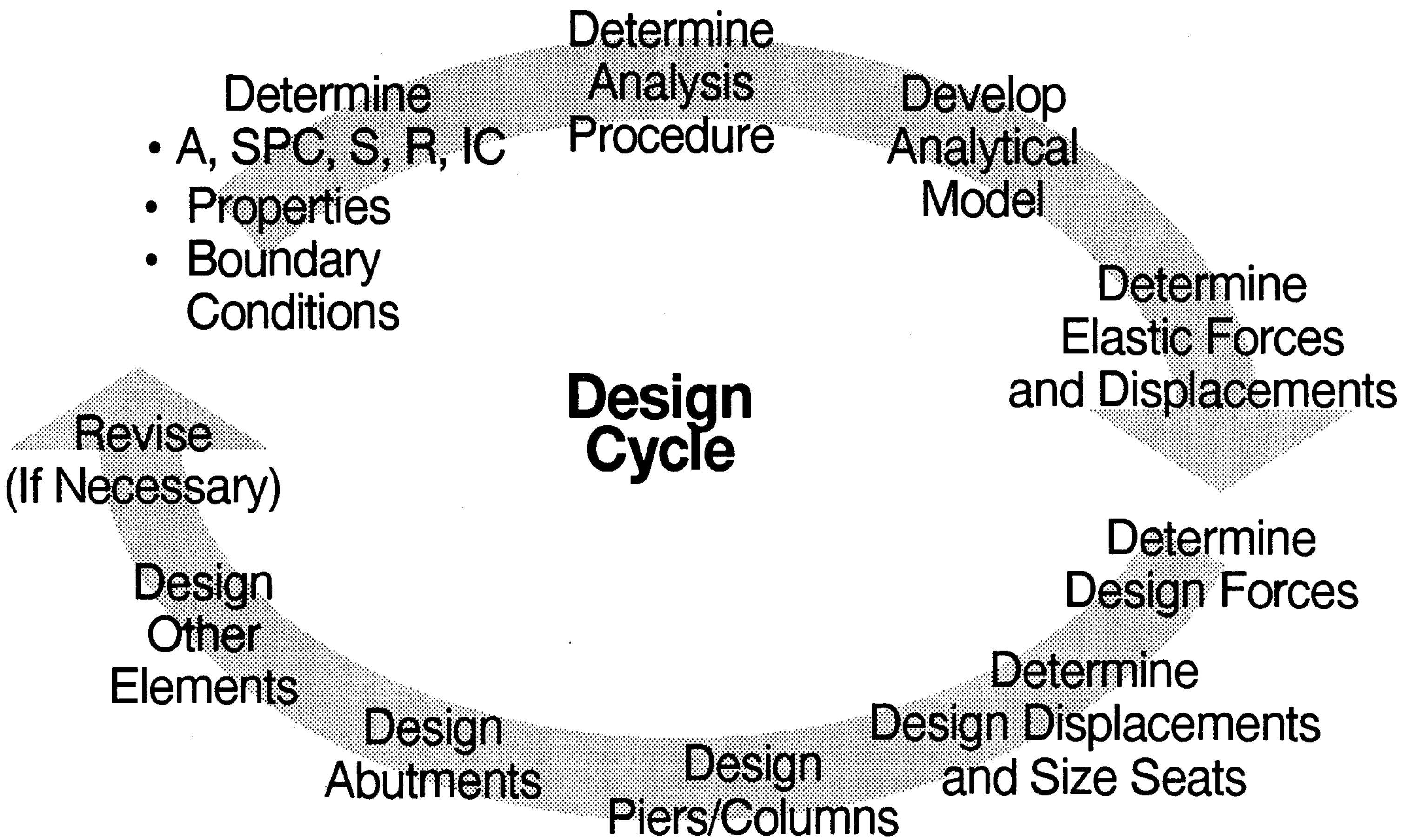


Limited Ductility — $R = 2$

Elastic vs. Design vs. Actual Forces



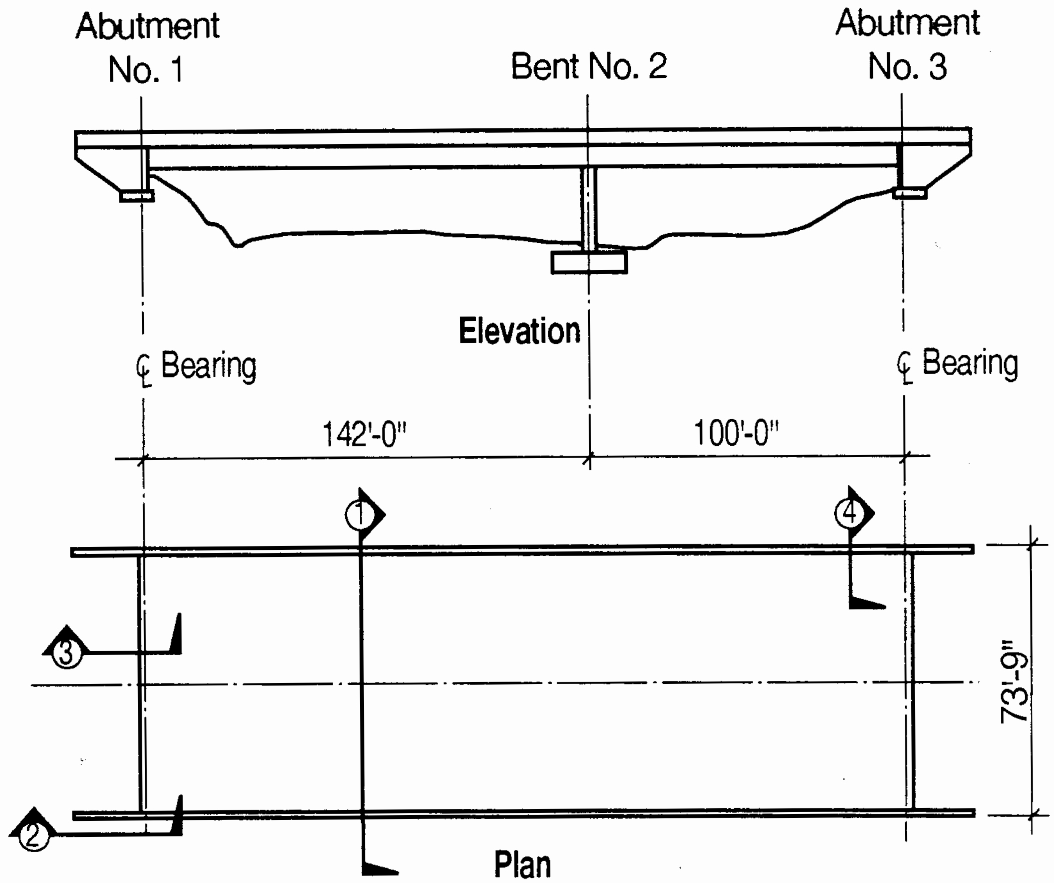
AASHTO Division 1-A



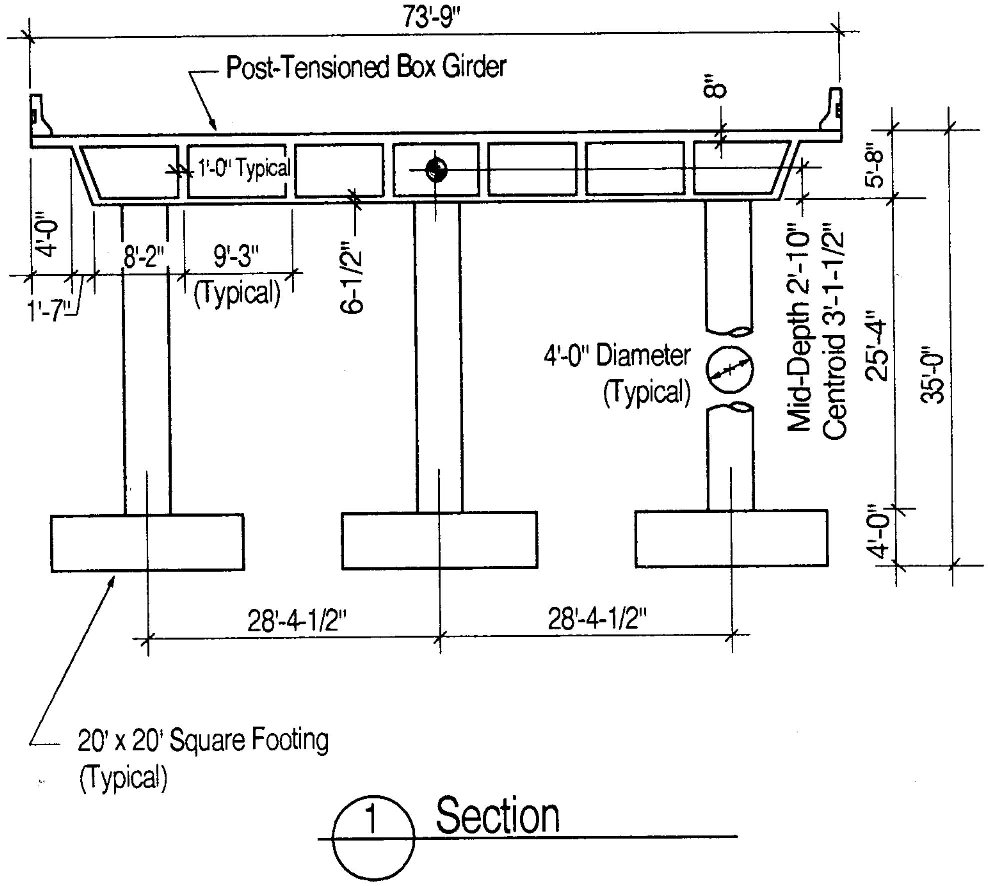
Session 3
Example Analysis / Two-Span Bridge

- Bridge Layout and Basic Data
- Behavior
- Mathematical Model
- Earthquake Direction
- Longitudinal Analysis
- Transverse Analysis

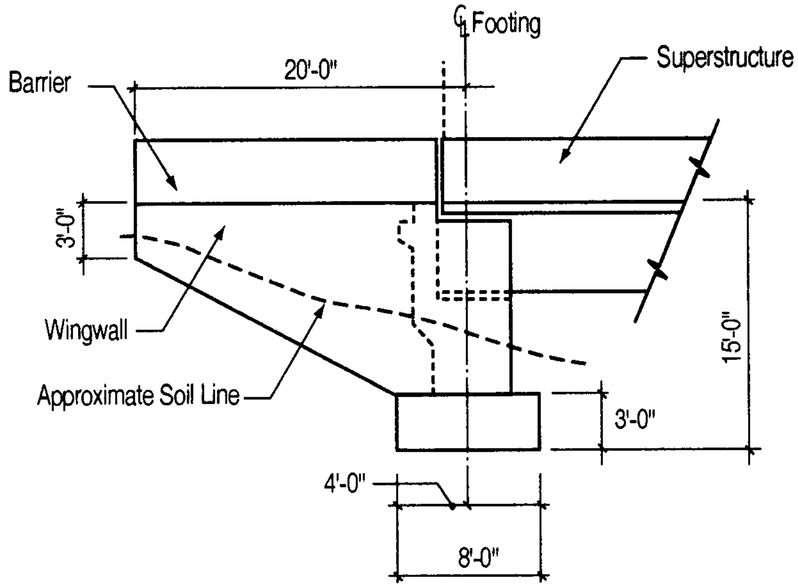
Layout / Plan and Elevation



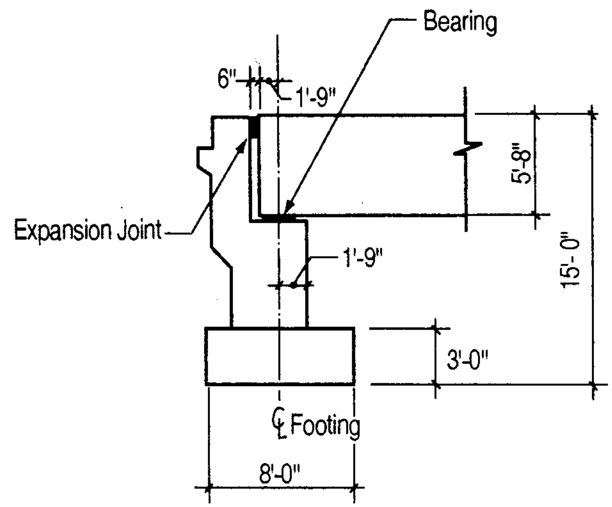
Layout / Preliminary Bent Details



Bridge Layout / Abutment Details

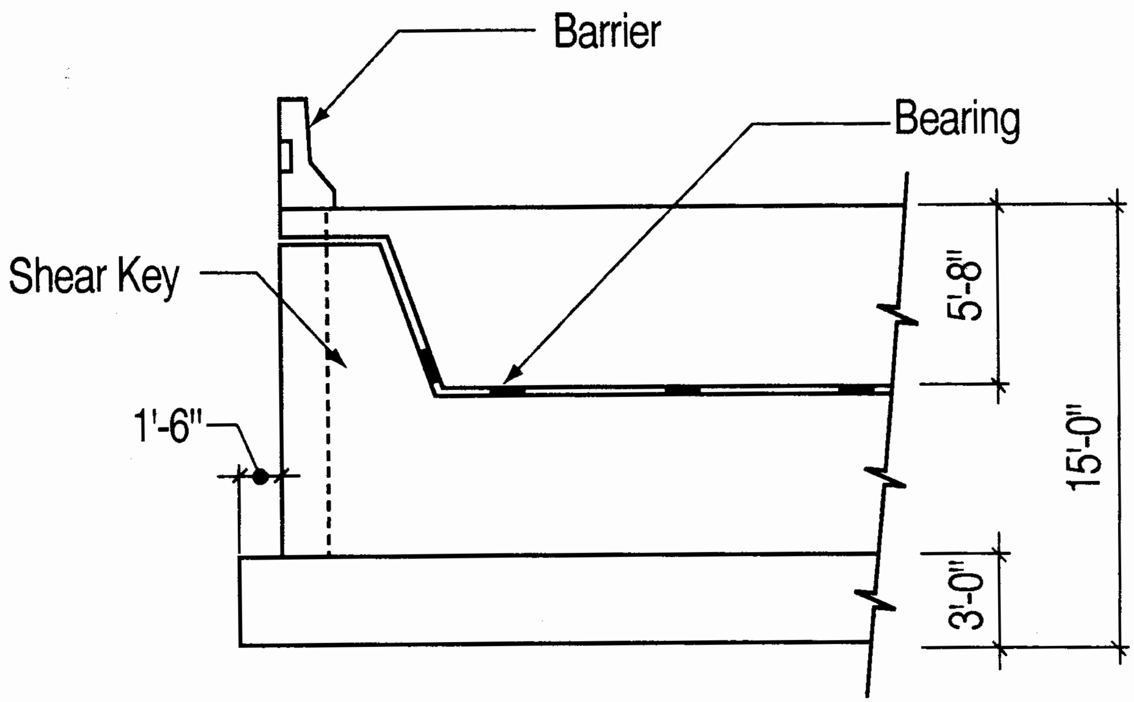


2 Section



3 Section

Layout / Shear Key at Abutments



4 Section

Basic Data for Bridge

- From AASHTO Map: **A = 0.28g** (Interpolation Permitted)
- Soil Is **250 ft Deep Glacial Sand and Gravel**
- Bridge Is **Not Essential**

Determine Seismic Performance Category

A = 0.28 g   IC = Not Essential

TABLE 1. Seismic Performance Category (SPC)

Acceleration Coefficient A	Importance Classification (IC) I	Importance Classification (IC) II
$A \le 0.09$	**A**	**A**
$0.09 < A \le 0.19$	**B**	**B**
$0.19 < A \le 0.29$	**C**	**C**
$0.29 < A$	**D**	**C**

SPC C

Determine Soil Site Coefficient

Four Basic Types of Soil:

I. Rock or Stiff Soil < 200 ft Thick
II. Deep Stiff Soil > 200 ft Thick
III. Soft to Medium Clay and Sands > 30 ft Thick
IV. Soft Clay or Silt > 40 ft Thick

TABLE 2. Site Coefficient

	Soil Profile Type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

S = 1.2

Response Modification Factors

Intermediate Substructure = Multiple Column Bent

TABLE 3. Response Modification Factor (R)

Substructure	R	Connections	R
Wall-Type Pier	2	Superstructure to Abutment	0.8
Reinforced Concrete Pile Bents		Expansion Joints within a Span of the Superstructure	0.8
a. Vertical Piles Only	3	Columns, Piers, or Pile Bents to Cap Beam or Superstructure	1.0
b. One or More Batter Piles	2	Columns or Piers to Foundations	1.0
Single Columns	3		
Steel or Composite Steel and Concrete Pile Bents			
a. Vertical Piles Only	5		
b. One or More Batter Piles	3		
Multiple Column Bent	5		

Determine Analysis Procedure

- Straight Alignment
- Span Length Ratio:

$$\frac{142}{100} = 1.42 < 3$$

- Bent Stiffness Ratio: NA

TABLE 5. Regular Bridge Requirements

Parameter	Value				
Number of Spans	**2**	**3**	**4**	**5**	**6**
Maximum Subtended Angle (Curved Bridge)	90° ✓	90°	90°	90°	90°
Maximum Span Length Ratio from Span-to-Span	3 ✓	2	2	1.5	1.5
Maximum Bent/Pier Stiffness Ratio from Span-to-Span (Excluding Abutments)	-- ✓	4	4	3	2

Regular

Determine Analysis Procedure (continued)

TABLE 5. Minimum Analysis Requirements

Seismic Performance Category	Regular Bridges with 2 Through 6 Spans	Not Regular Bridges with 2 or More Spans
A	Not Required	Not Required
B, C, D	Use Procedure 1 or 2	Use Procedure 3

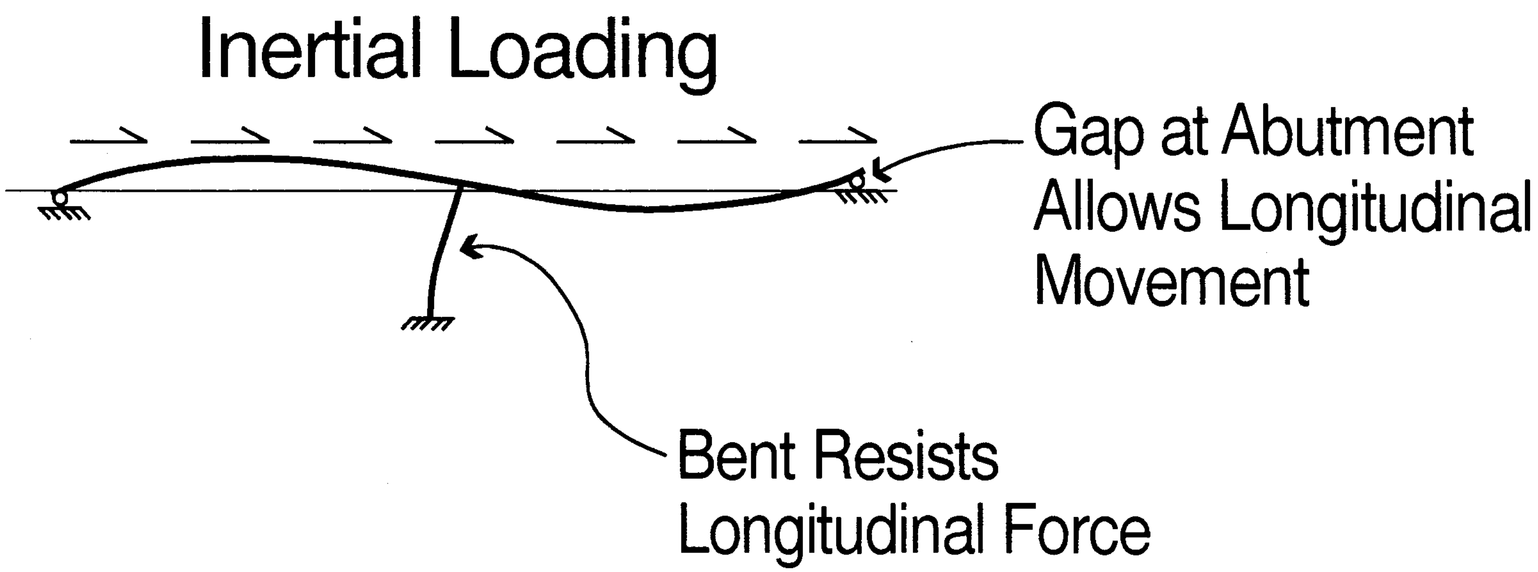
May Use:

1. Uniform Method
   Longitudinal
2. Single-Mode Spectral
   Transverse
3. Multimode Spectral
4. Time History

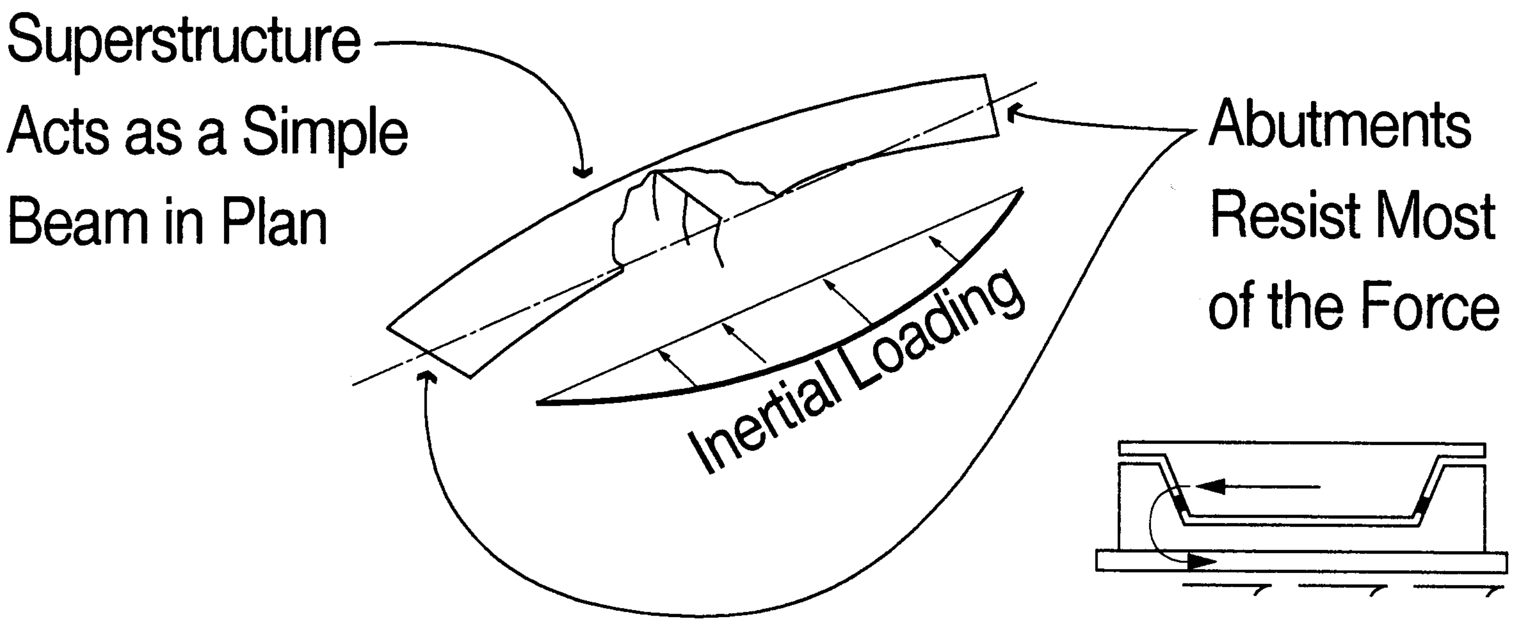
Session 3

- Bridge Layout and Design
- **Behavior**
- Mathematical Model
- Earthquake Direction
- Longitudinal Analysis
- Transverse Analysis

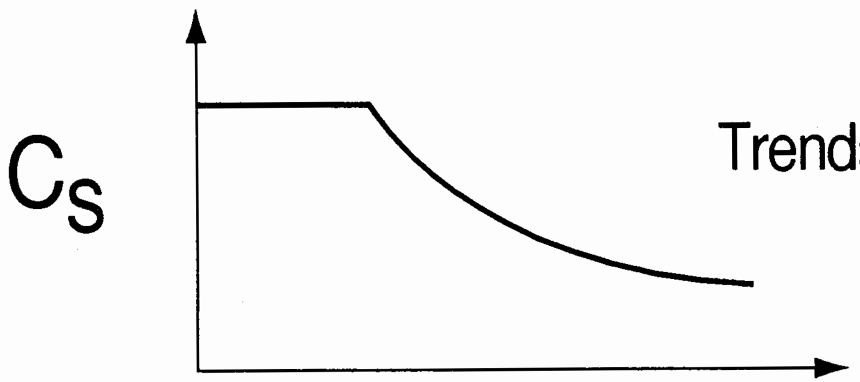
Longitudinal Lateral Load Behavior



Transverse Lateral Load Behavior



Bounding the Response



Trends: K ↑ T ↓ C_S ↑ V ↑

also Δ ↓

$$T = 2\pi \sqrt{\frac{W}{gK}}$$

Note

- **Total Base Shear:** V Is Proportional to $C_S \bullet W$

Bounding the Response (continued)

Implications

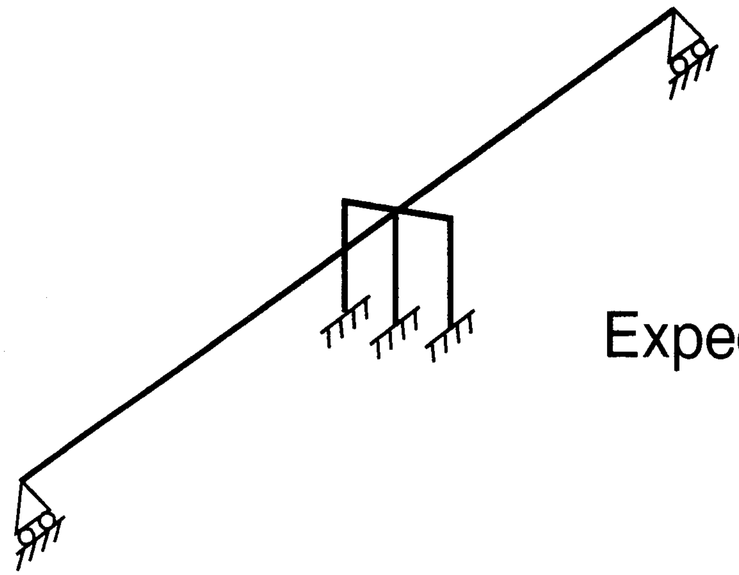
In General, Stiffening the Structure Leads to

- Larger Forces
- Smaller Displacements

Conversely, Softening the Structure Leads to

- Smaller Forces
- Larger Displacements

Alternatives to Consider / No. 1

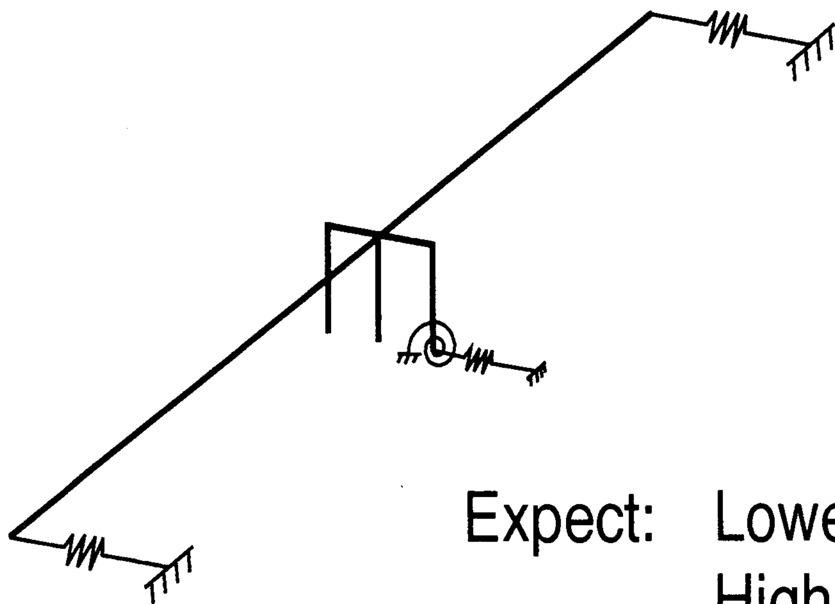


'Fixed Supports'

Expect: Higher Forces
Lower Displacements

Use for Analysis to Get
Upper-Bound for Elastic Forces

Alternatives to Consider / No. 2



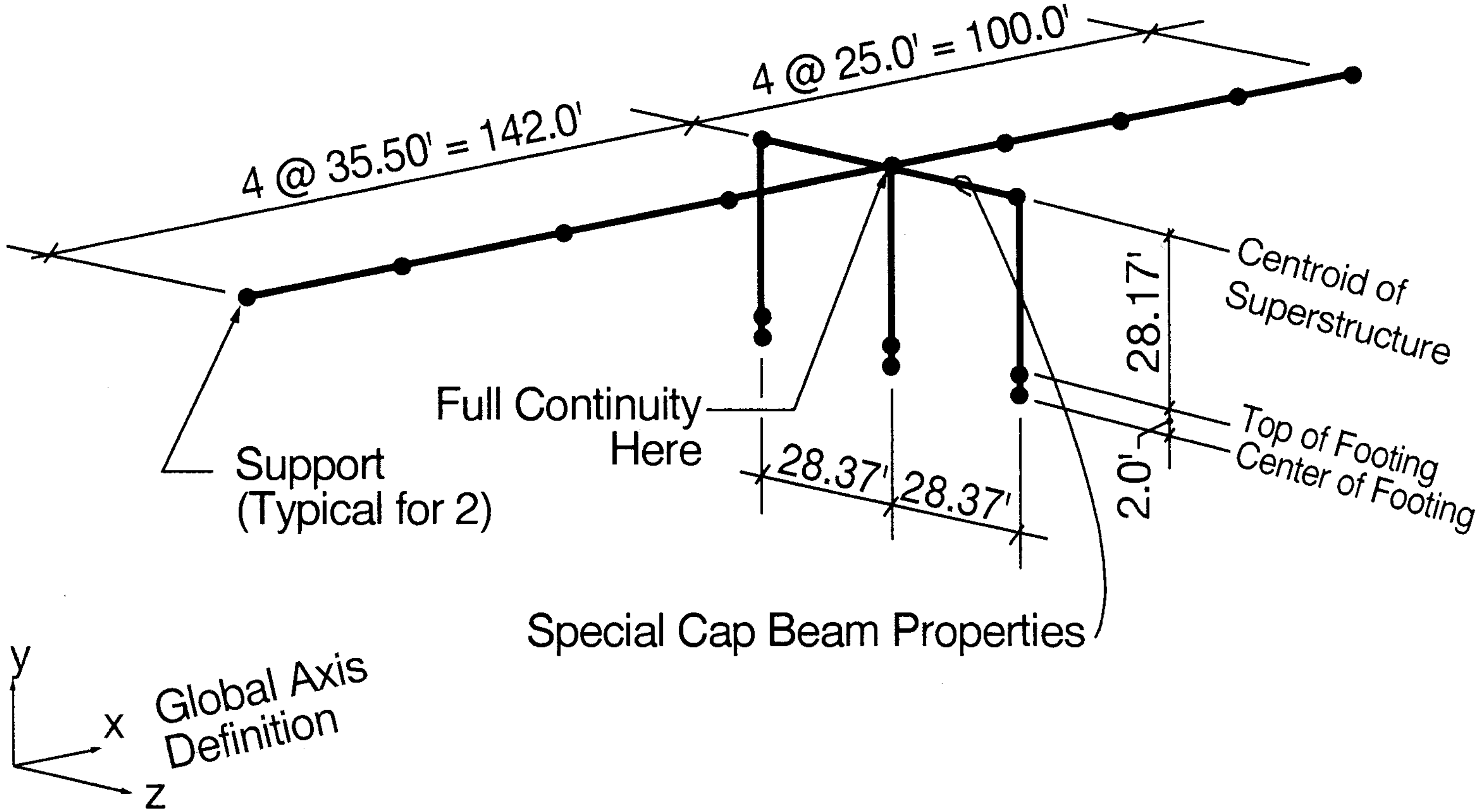
'Spring Supports'

Expect: Lower Forces
Higher Displacements

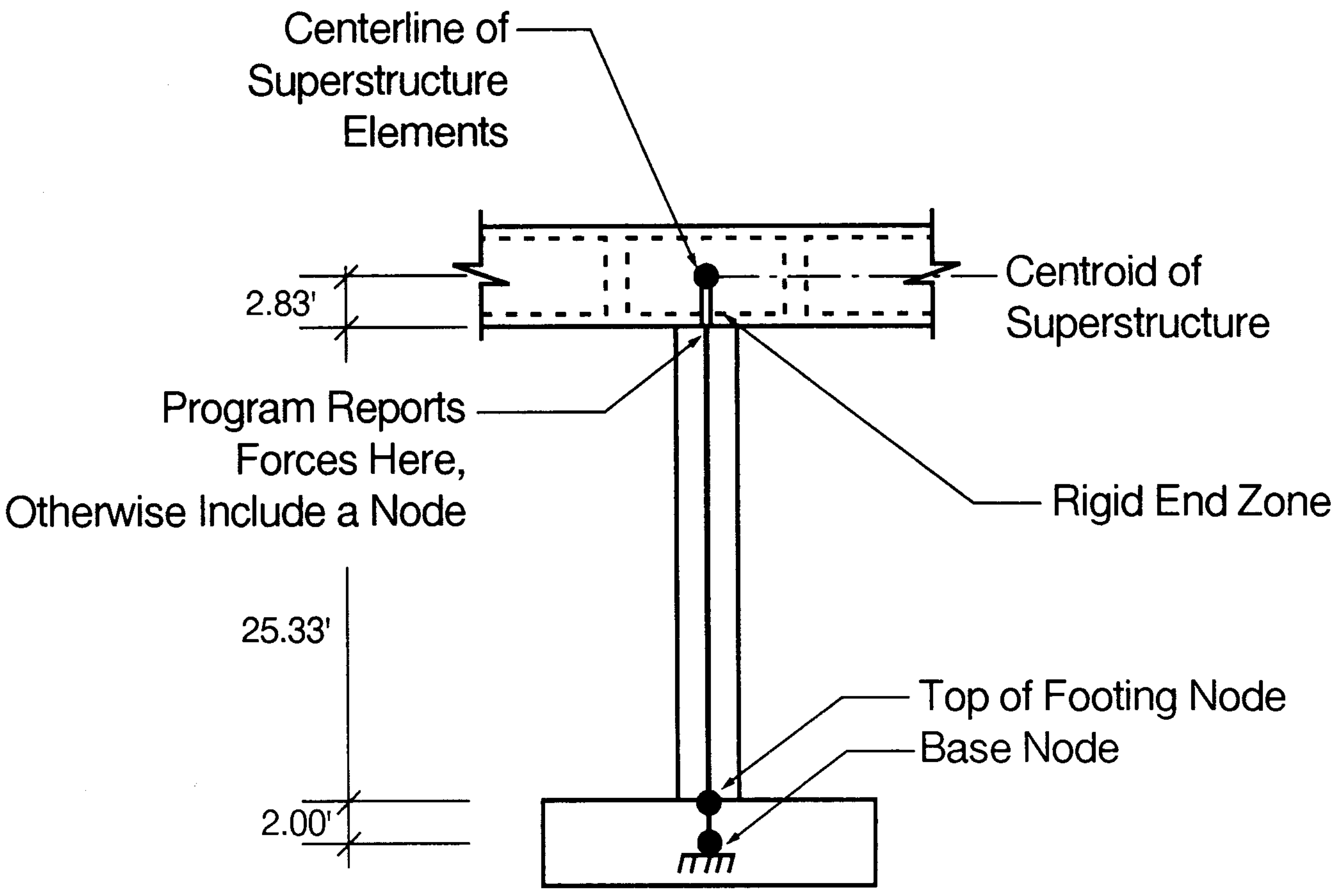
Session 3

- Bridge Layout and Design
- Behavior
- **Mathematical Model**
- Earthquake Direction
- Longitudinal Analysis
- Transverse Analysis

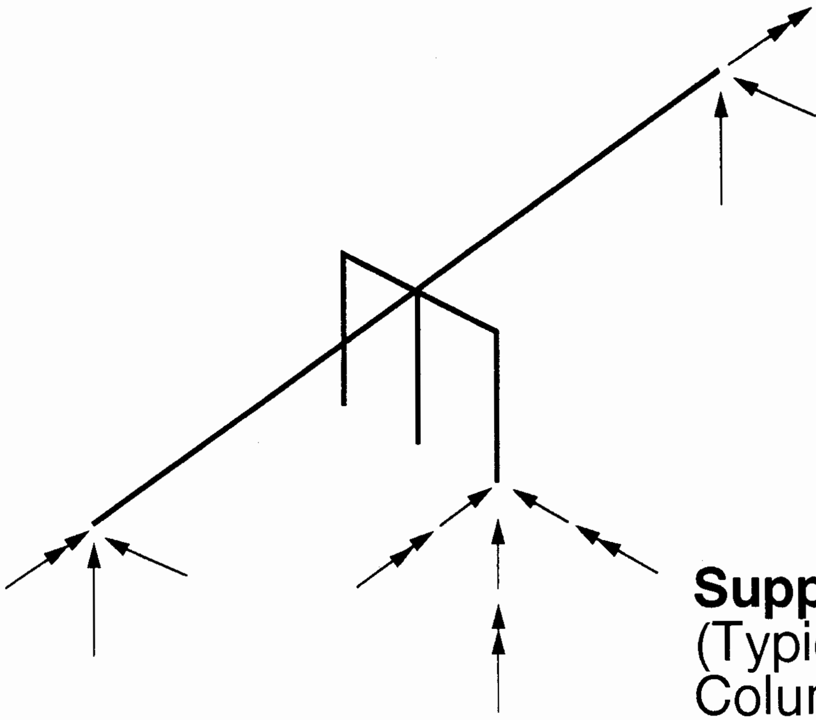
Mathematical Model for Analysis



Column and Footing Element Geometry



Support Conditions



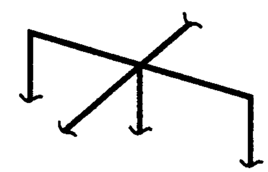
Support Restraint
(Typical for All Three Columns)

Vector Arrows Indicate Support Restraint in the Direction Shown

Properties

$f'_c = 4000$ psi $E = 518{,}400$ ksf

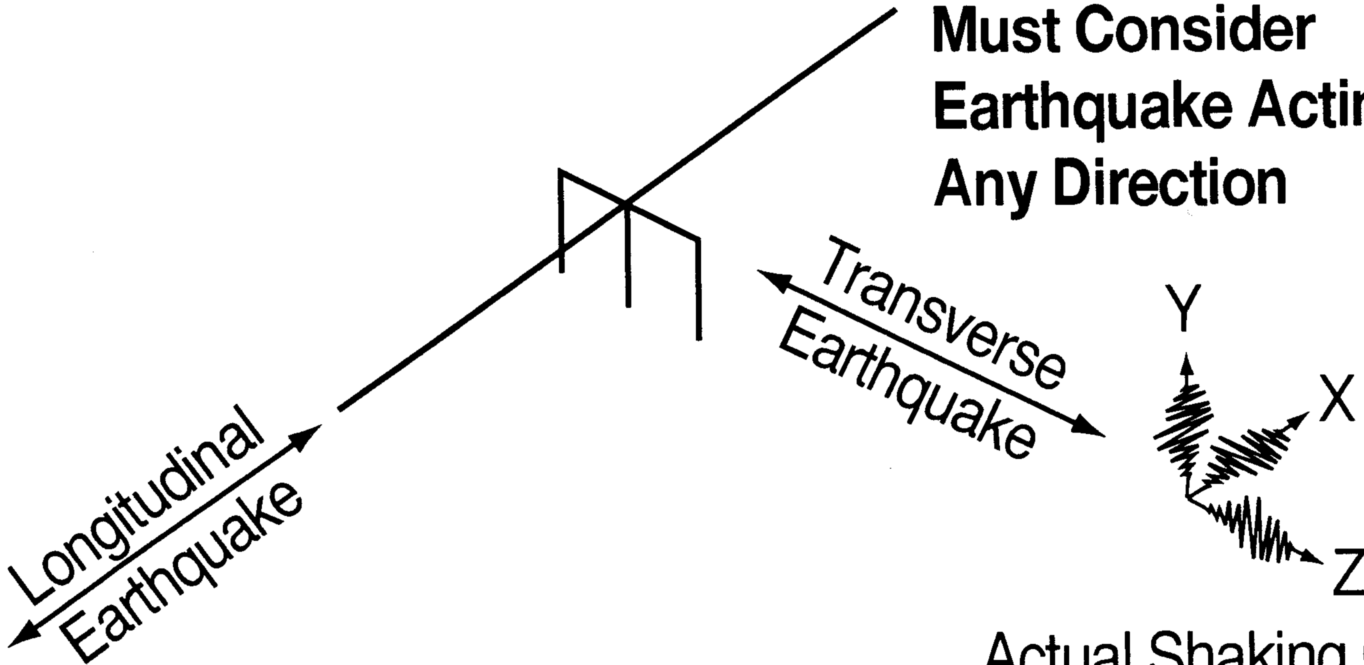
Superstructure	Column	Cap Beam
$A = 120\ ft^2$	$A = 12.6\ ft^2$	$A = 25\ ft^2$
$I_{str} = 51{,}000\ ft^4$	$I = 12.6\ ft^4$	$I_{str} = 10^7\ ft^4$
$I_{weak} = 575\ ft^4$		$I_{weak} = 10^7\ ft^4$

Properties for Lateral Analysis Only 

Session 3

- Bridge Layout and Design
- Behavior
- Mathematical Model
- **Earthquake Direction**
- Longitudinal Analysis
- Transverse Analysis

Earthquake Direction



Must Consider Earthquake Acting in Any Direction

Actual Shaking Occurs in All Three Directions

Directional Combinations for Loading

Two Analyses:

- Orthogonal Horizontal Directions
- Actual Earthquake Attack May Be from Any Direction
- Maximum Inputs **Do Not** Occur Simultaneously in Each Direction

Directional Combinations for Loading (continued)

- **Load Combinations**

  LC1 = 100% Longitudinal + 30% Transverse

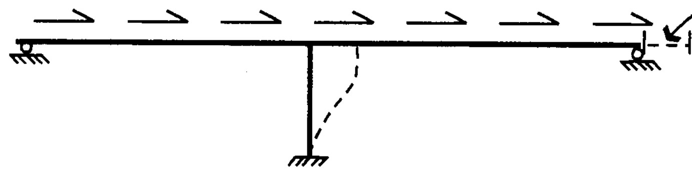
  LC2 = 100% Transverse + 30% Longitudinal

Session 3

- Bridge Layout and Design
- Behavior
- Mathematical Model
- Earthquake Direction
- **Longitudinal Analysis**
- Transverse Analysis

Uniform Load Method / Section 4.3 (1-A)

- **Step 1** $P_o = 100$ kip/ft (Arbitrary) $\Delta_{max} = 2.55$ ft (Hand or Computer Calculation)



- **Step 2** Stiffness $K = \dfrac{P_o L}{\Delta_{max}} = \dfrac{100(242)}{2.55} = 9486$ kip/ft

  Weight $W = 4876$ kip

- **Step 3** $T = 2\pi\sqrt{\dfrac{W}{gK}} = 2\pi\sqrt{\dfrac{4876}{32.2\ (9486)}} = 0.79$ sec

Uniform Load Method (continued)

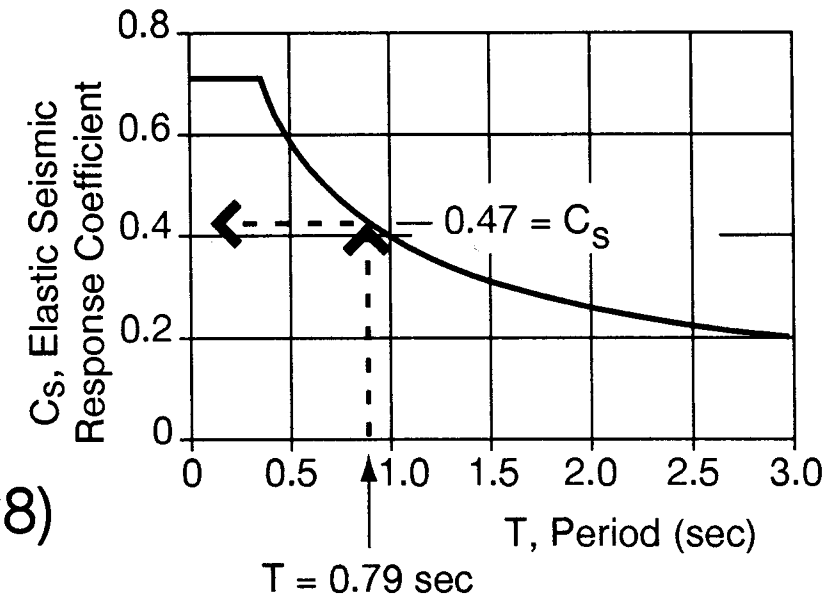
- **Step 4**

$$C_S = \frac{1.2AS}{T^{2/3}} \leq 2.5\,A$$

$$C_S = \frac{1.2(0.28)1.2}{(0.79)^{2/3}} \leq 2.5\,(0.28)$$

$$C_S = 0.47 \leq 0.70$$

Controls



Uniform Load Method (continued)

- **Step 4** (continued)

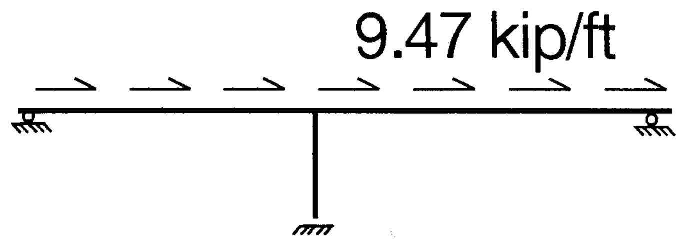
$$P_e(x) = \frac{C_s W}{L} = \frac{0.47\ (4876)}{242}$$

$$P_e(x) = 9.47\ \text{kip/ft}$$

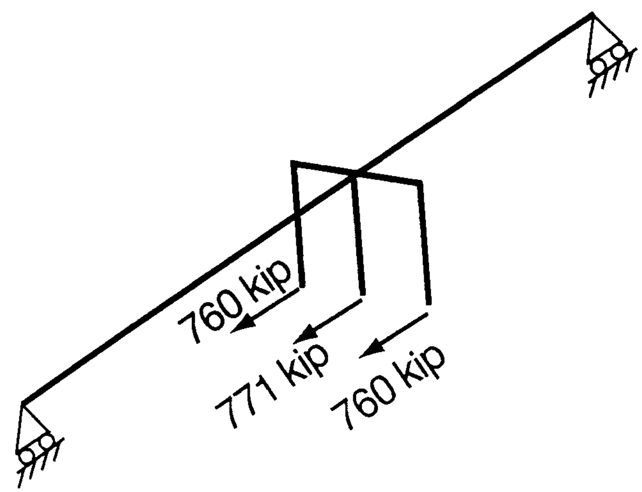
Earthquake Load

Uniform Load Method (continued)

- **Step 5**



			Forces and Moments				
			Longitudinal		Transverse		
			Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)	Axial (kips)
Abutment No. 1			0	0	0	0	105
Bent No. 2	Center	Top	771	9978	0	0	35.5
		Bottom	771	9566	0	0	35.5
	Outboard	Top	760	9790	0	0	35.5
		Bottom	760	9481	0	0	35.5
Abutment No. 3			0	0	0	0	211

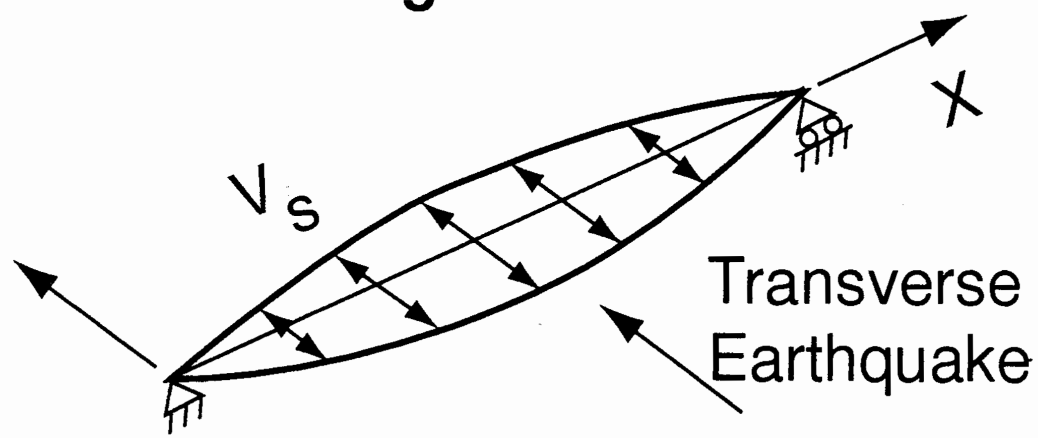


Session 3

- Bridge Layout and Design
- Behavior
- Mathematical Model
- Earthquake Direction
- Longitudinal Analysis
- **Transverse Analysis**

Single-Mode Spectral Method / 4.4 (1-A)

Concept: Structure Responds in Single Vibration Mode



Shape ⊜ Deflection from Uniform Lateral Load

Single-Mode Spectral Method Steps

1. Apply Uniform Load, P_o / Obtain Deflection, $V_s(x)$
2. Calculate Modal Weighting Factors

$$\alpha = \int_0^L V_s(x)dx$$

$$\beta = \int_0^L w_s(x)V_s(x)dx$$

$$\gamma = \int_0^L w_s(x)V_s^2(x)dx$$

3. Calculated Period, $T=2\pi\sqrt{\dfrac{\gamma}{P_o g \alpha}}$
4. Calculate Inertial Loading,

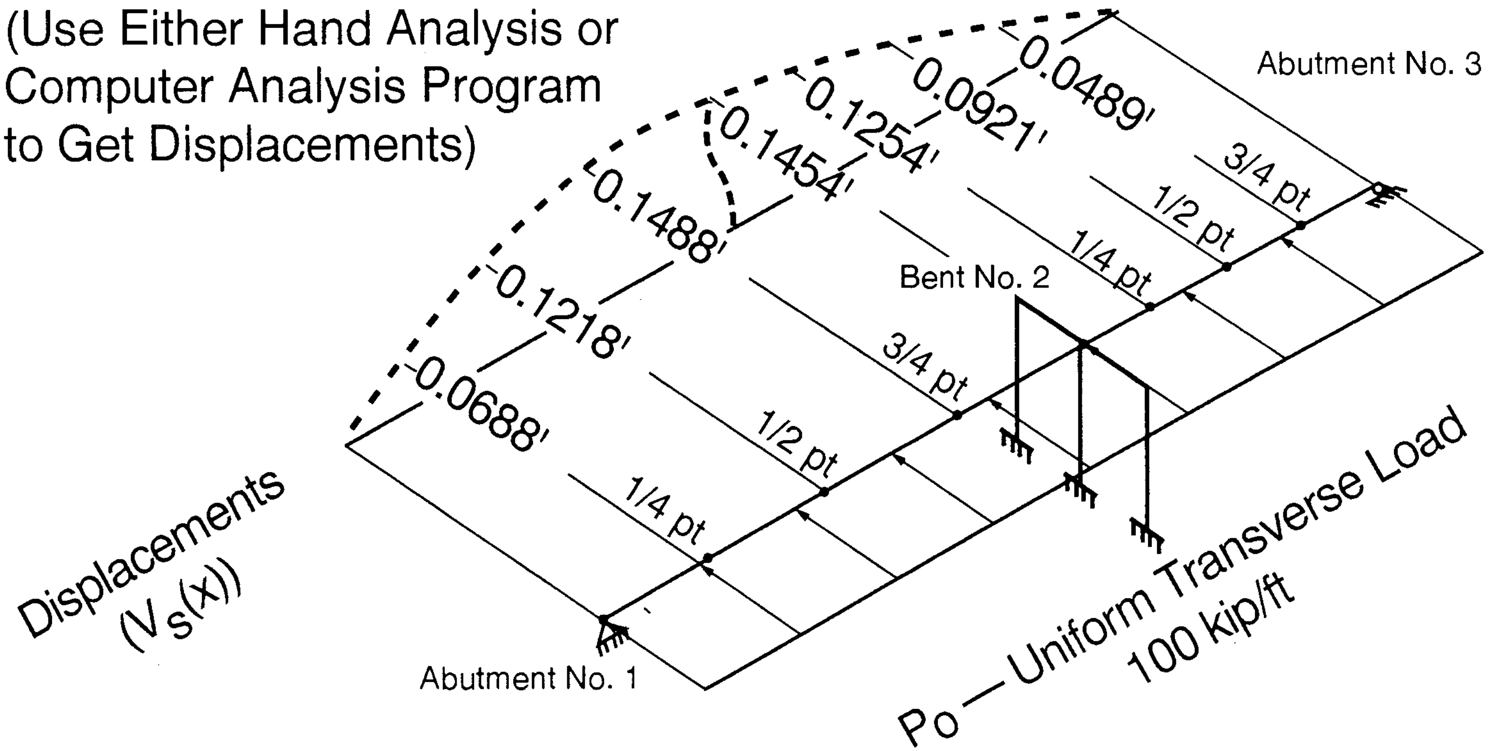
$$P_e(x) = \frac{\beta C_s}{\gamma} w(x)\, V_s(x)$$

5. Apply $P_e(x)$ / Find Forces, Displacements

Single-Mode Spectral Method / Step 1

- **Apply Uniform Load/Obtain Displacements**

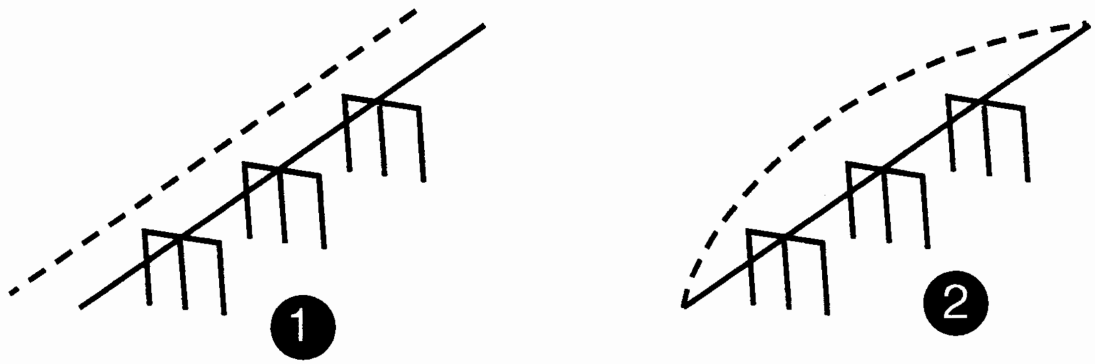
  (Use Either Hand Analysis or Computer Analysis Program to Get Displacements)



'Weighting Factors' α, β, γ / Step 2

Transverse Seismic Movement

(Hypothetical 4-Span Example)

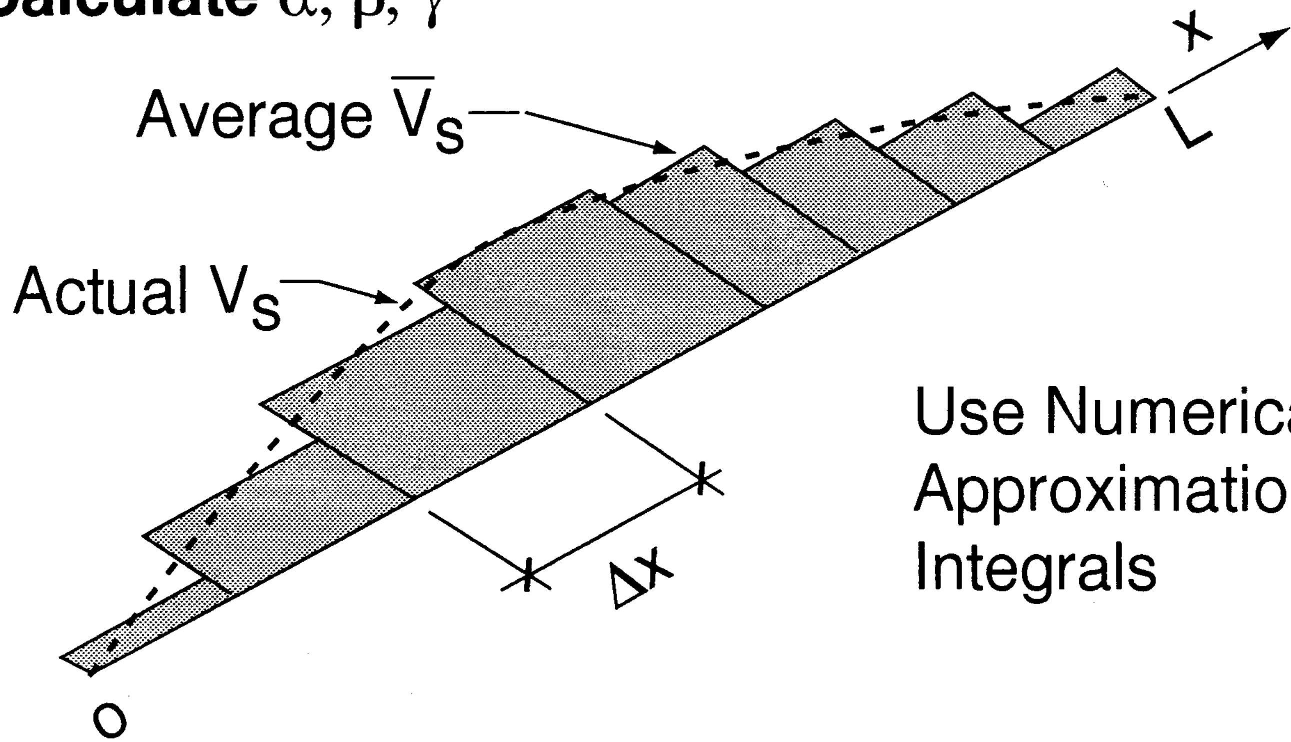


Weighting Factors Account for

- Resisting Elements (Piers, etc.) Deflect Differently
- Inertial Forces Vary in Accordance with Deflection

Single-Mode Spectral Method / Step 2

- **Calculate** α, β, γ

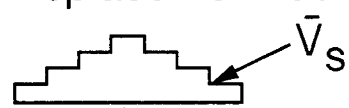


Use Numerical Approximation of Integrals

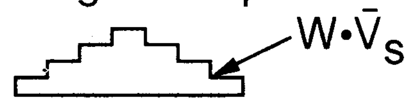
Single-Mode Spectral Method / Step 2 (continued)

		AASHTO eqn.
$\alpha = \int_0^L V_s dx \longrightarrow$	$\alpha = \Sigma \bar{V}_s \Delta x$	(4-5)
$\beta = \int_0^L w V_s dx \longrightarrow$	$\beta = \Sigma w \bar{V}_s \Delta x$	(4-6)
$\gamma = \int_0^L w V_s^2 dx \longrightarrow$	$\gamma = \Sigma w \bar{V}_s^2 \Delta x$	(4-7)

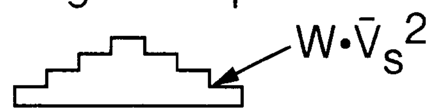
$\alpha =$ Area Under Displacement Curve



$\beta =$ Area Under Weight • Displacement



$\gamma =$ Area Under Weight • Displacement^2



Single-Mode Spectral Method / Step 2 (continued)

Assumptions:

P_o = 100.0 k/ft A = 0.28
g = 32.2 ft/sec^2 2.5*A = 0.70
w(x) = 20.1 k/ft S = 1.2

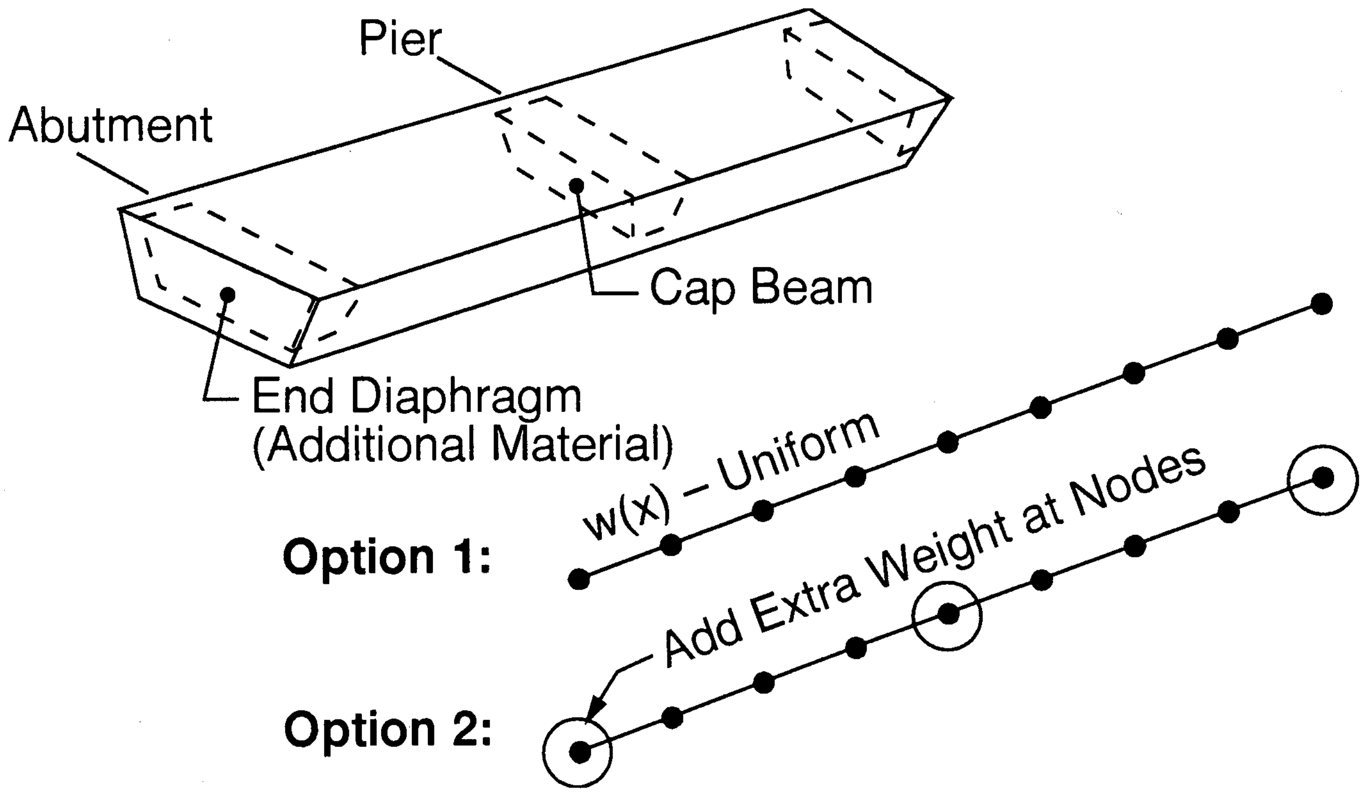
1	2	3	4	5	6	7	8
Location	Node Distance x (ft)	Tributary Length dx (ft)	Displ Due to Uniform Loading $v_s(x)$ (ft)	$\alpha(x)$ (ft^2)	$\beta(x)$ (k-ft)	$\gamma(x)$ (k-ft^2)	Equiv. Static EQ Loading $P_e(x)$ (k-ft)
Abut No. 1	0.0	0.0	0.0000				0.00
1/4 pt	35.5	35.5	0.0688	1.22	24.55	1.69	8.03
1/2 pt	71.0	35.5	0.1218	3.38	68.02	6.98	14.23
3/4 pt	106.5	35.5	0.1488	4.80	96.54	13.19	17.37
Bent No. 2	142.0	35.5	0.1454	5.22	104.94	15.43	16.97
1/4 pt	167.0	25.0	0.1252	3.38	67.99	9.25	14.62
1/2 pt	192.0	25.0	0.0921	2.72	54.61	6.07	10.76
3/4 pt	217.0	25.0	0.0489	1.76	35.44	2.73	5.71
Abut No. 3	242.0	25.0	0.0000	0.61	12.29	0.60	0.00
Sum =		242.0		23.10	464.38	55.96	

Weight
Include Structure, Barriers
Overlay, Diaphragms, etc.

(Either Lump or Spread Evenly)

$\alpha = 23.10\ ft^2$
$\beta = 464.4$ kip ft
$\gamma = 55.96$ k ft^2

Weight Distribution / Step 2



Single-Mode Spectral Method / Step 3

- **Calculate Period T**

$$T = 2\pi\sqrt{\frac{\gamma}{P_o g \alpha}} = 2\pi\sqrt{\frac{55.96}{100\,(32.2)23.10}} \qquad \text{Eqn (4-8)}$$

$$T = 0.17 \text{ sec}$$

Units:

$$T = 2\pi\sqrt{\frac{\text{kip ft}^2}{\left(\frac{\text{kip}}{\text{ft}}\right)\left(\frac{\text{ft}}{\text{sec}^2}\right)\left(\text{ft}^2\right)}} = \sqrt{\text{sec}^2} = \text{sec}$$

Single-Mode Spectral Method / Step 4

- **Calculate Equivalent Static Earthquake Loading**

Elastic Seismic Response Coefficient, C_s

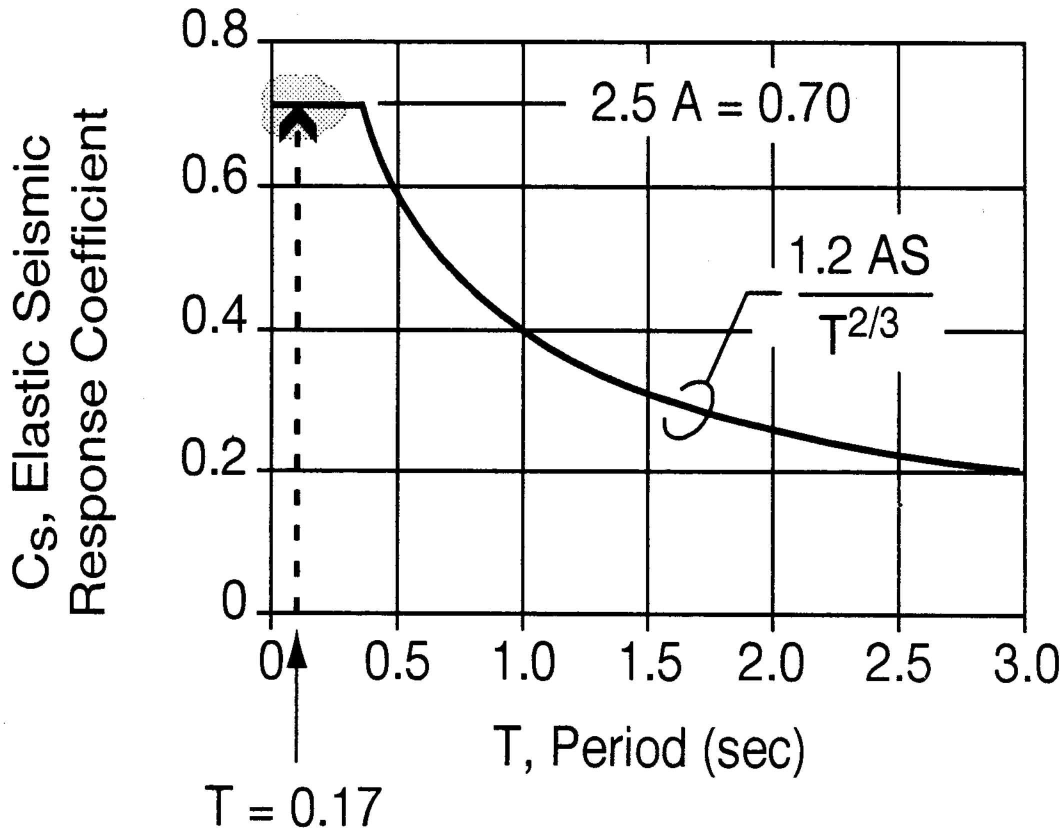
$$C_s = \frac{1.2AS}{T^{2/3}} \leq 2.5\,A$$

$$C_s = \frac{1.2(0.28)1.2}{(0.17)^{2/3}} \leq 2.5\,(0.28)$$

$$C_s = 1.30 \leq 0.70$$

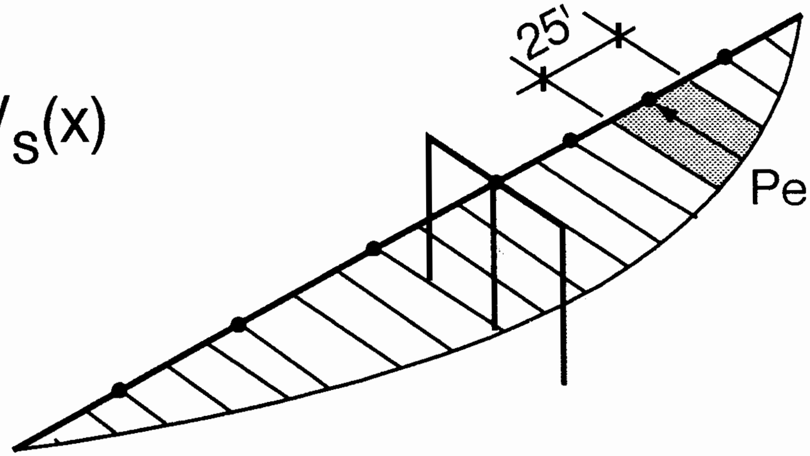
Controls

Single-Mode Method / Step 4 (continued)



Single-Mode Method / Step 4 (continued)

$$P_e(x) = \frac{\beta C_s}{\gamma} \, w(x) V_s(x)$$

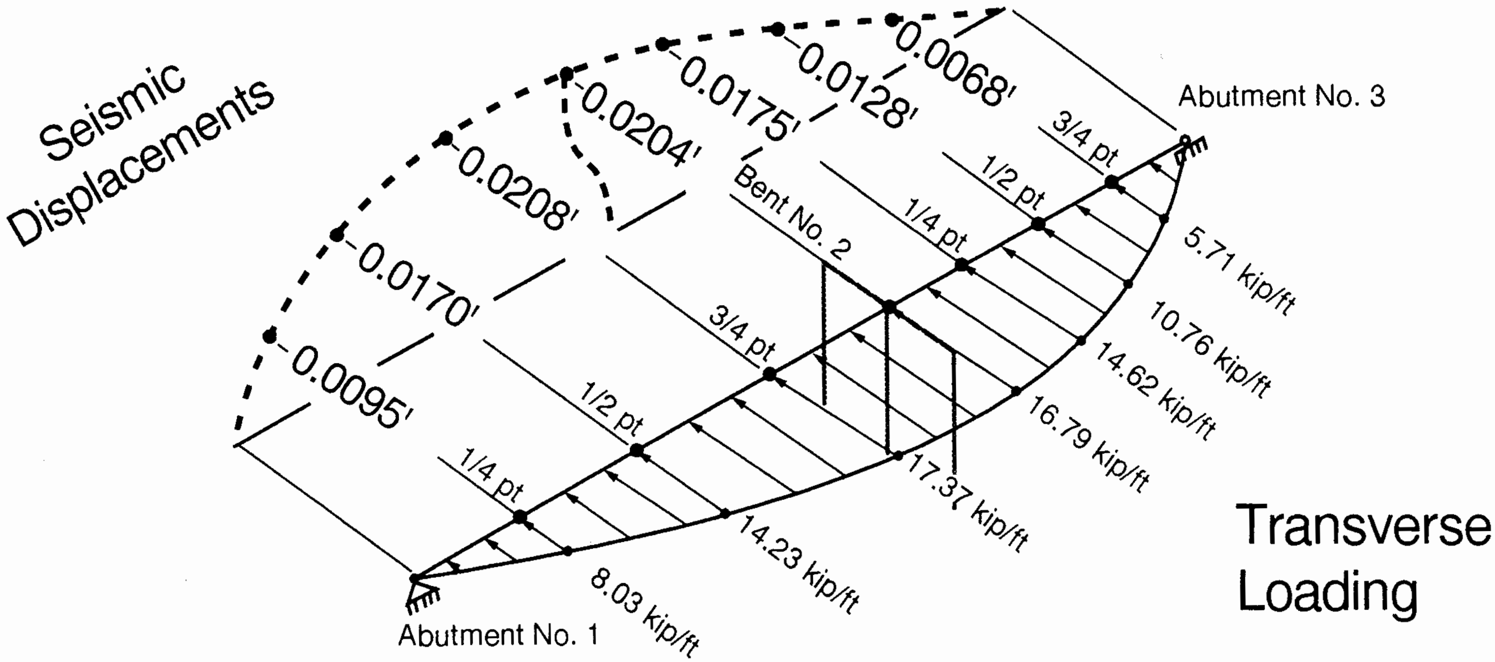


$$P_e = \frac{464.4}{55.96} (0.70)\, 20.1 (0.0921) = 10.76 \text{ kip/ft}$$ (Load Intensity)

$$P_e = 10.76\,(25) = 269 \text{ kip}$$ (Concentrated Load at Node)

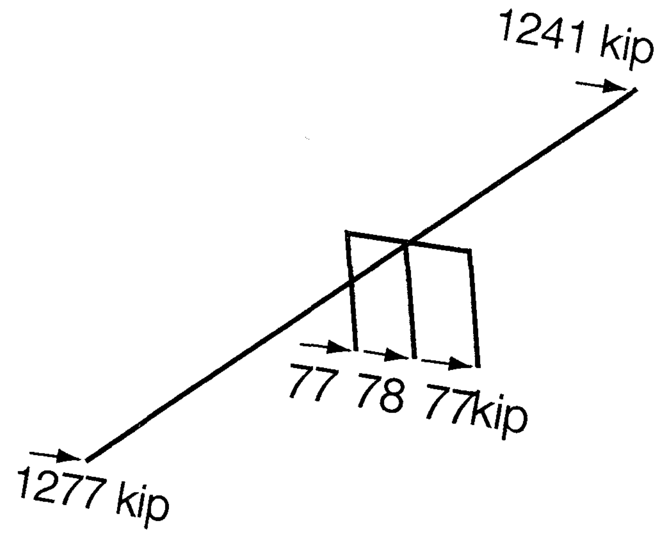
Single-Mode Spectral Method / Step 5

- **Apply $P_e(x)$ / Find Forces, Displacements**



Transverse Loading Results

			Forces and Moments				
			Longitudinal		Transverse		
			Shear (kips)	Moment (kip-ft)	Shear (kips)	Moment (kip-ft)	Axial (kips)
Abutment No. 1			0	0	1277	583	0
Bent No. 2	Center	Top	0	0	77.8	1062	0
		Bottom	0	0	77.8	910	0
	Outboard	Top	8.1	110	77.2	1053	42.5
		Bottom	8.1	94.7	77.2	902	42.5
Abutment No. 3			0	0	1241	828	0



Session 4
Example Design of Two-Span Bridge

- **Elastic Forces ⟶ Design Forces**
  (Including Column Flexural Design)
- **Design Columns**
- **Design Column Footings**
- **Abutment Issues**

From Elastic Seismic Forces To Design Forces

Directional LC1 and LC2 Combinations

LC1 = 100% Longitudinal Earthquake + 30% Transverse Earthquake

LC2 = 100% Transverse Earthquake + 30% Longitudinal Earthquake

↓

Modified Seismic Forces

Members $EQM_1 = LC1/R$

Foundations $EQF_1 = LC1$

↓

Modified Design Forces

Group Load = 1.0 (D + B + SF + E +EQM)

↓

(continued)

From Elastic Seismic Forces To Design Forces

SPC C and D

Column Flexural Design — Select Longitudinal Reinforcement

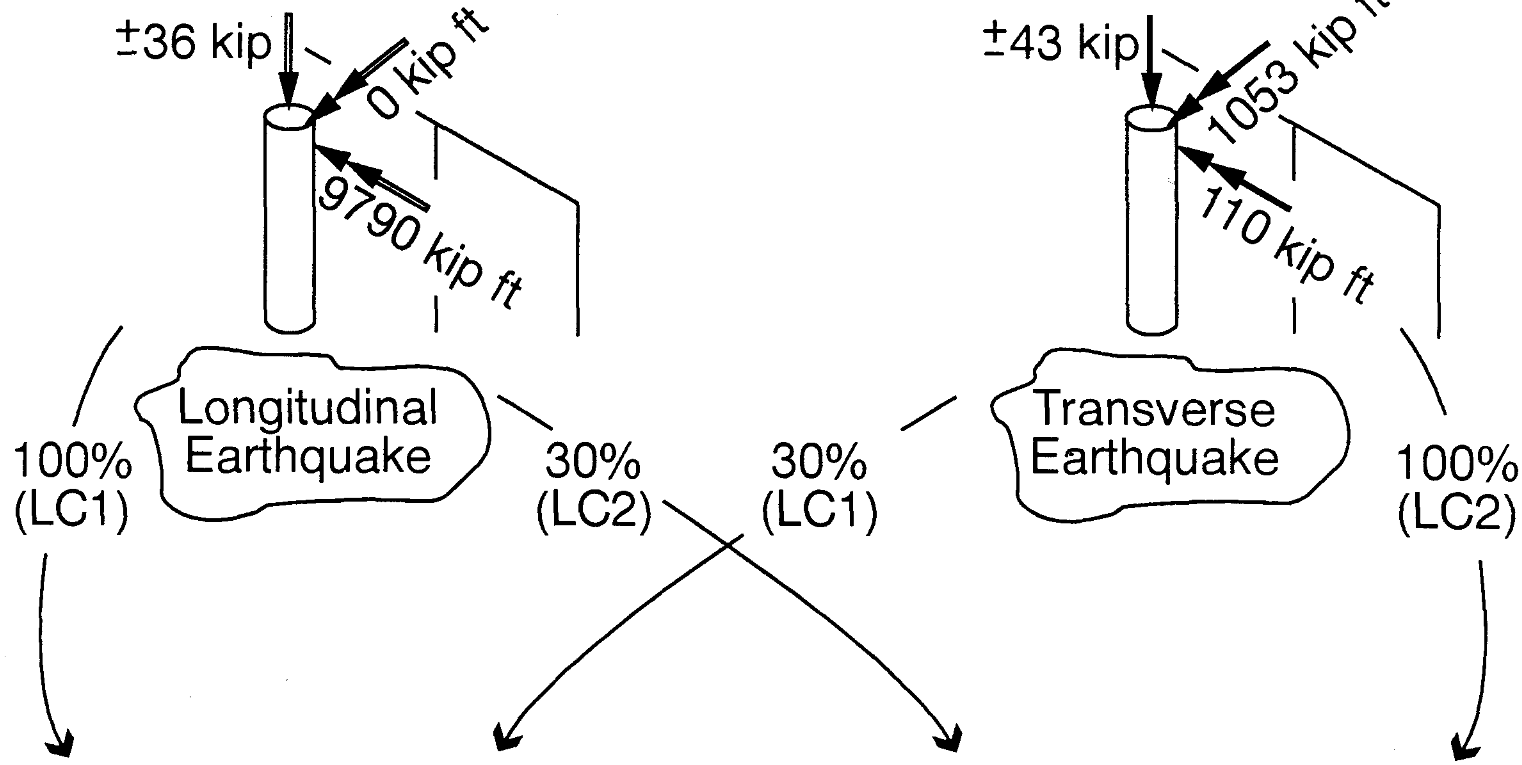
↓

Plastic Hinging Forces — Transverse and Longitudinal Directions; Realistic Member Strengths

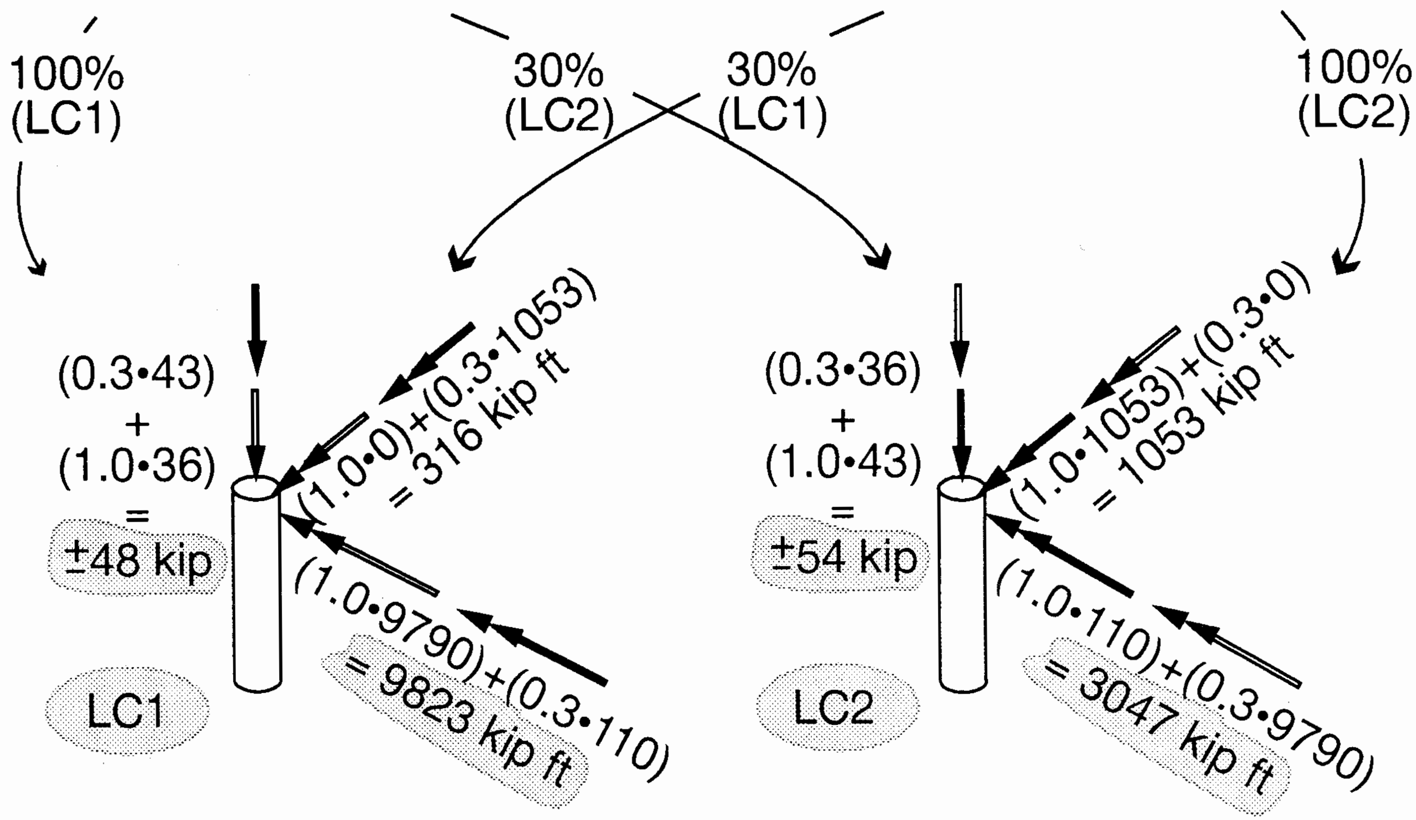
↓

Controlling Design Forces — Elastic vs. Modified vs. Plastic

Orthogonal Seismic Force Combination



Orthogonal Seismic Force Combination



Determine Modified Seismic Forces

Exterior Column

Top

$$EQM_1 = \frac{LC1}{R}$$

$R = 5$ Multiple Column Bent Moments

$$EQM_1 = \begin{cases} M_T = 316/5 = 63 \text{ kip ft} \\ M_L = 9823/5 = 1965 \text{ kip ft} \\ P = \pm 48/1 = \pm 48 \text{ kip} \end{cases}$$

$$EQM_2 = \begin{cases} M_T = 211 \text{ kip ft} \\ M_L = 609 \text{ kip ft} \\ P = \pm 54 \text{ kip} \end{cases}$$

(continued)

Use R = 1 for Axial Load and Shear

Determine Modified Seismic Forces (continued)

Bottom
(Modified Forces for Foundation)

$$EQF = \frac{LC1}{1.0}$$

R = 1 Footing

SPC C and D

$$EQF_1 = \begin{cases} M_T = 271 \text{ kip ft} \\ M_L = 9509 \text{ kip ft} \end{cases}$$

$$EQF_2 = \begin{cases} M_T = 902 \text{ kip ft} \\ M_L = 2939 \text{ kip ft} \end{cases}$$

Combine into Group Load to Get Modified Design Forces

0 0 0 ...for this Example

$$\text{Group Load} = 1.0\,(D + \cancelto{0}{B} + \cancelto{0}{SF} + \cancelto{0}{E} + EQM)$$

Substitute EQF for Foundations

Replaces Division I, Group VII

Combine into Group Load to Get Modified Design Forces (continued)

Group Load = D + EQM

Exterior Column Top

LC1

$M_T = 63$ kip ft

$M_L = 366 + 1965 = 2331$ kip ft

$P_{max} = 1098 + 48 = 1146$ kip

$P_{min} = 1098 - 48 = 1050$ kip

LC2

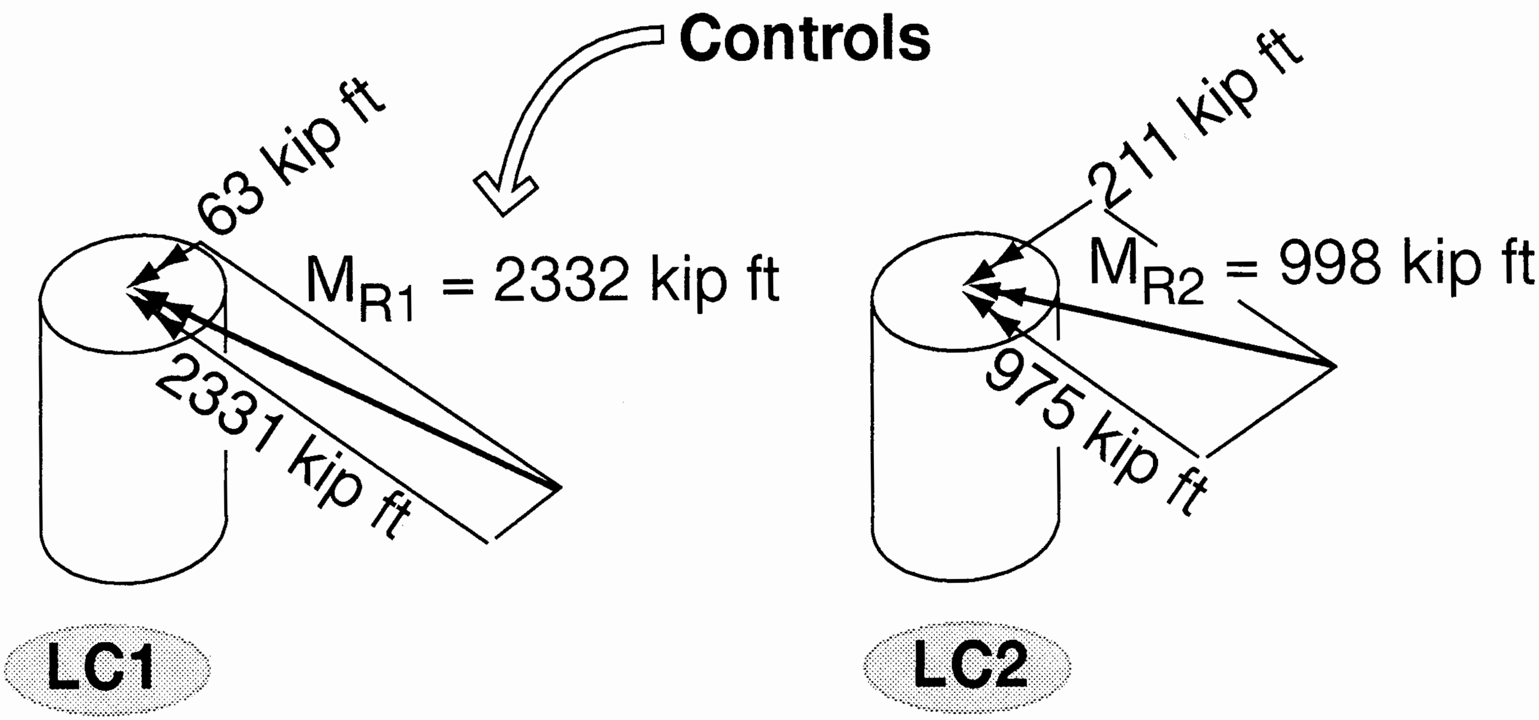
$M_T = 211$ kip ft

$M_L = 975$ kip ft

$P_{max} = 1152$ kip

$P_{min} = 1044$ kip

Determine Controlling Moment for Flexural Design



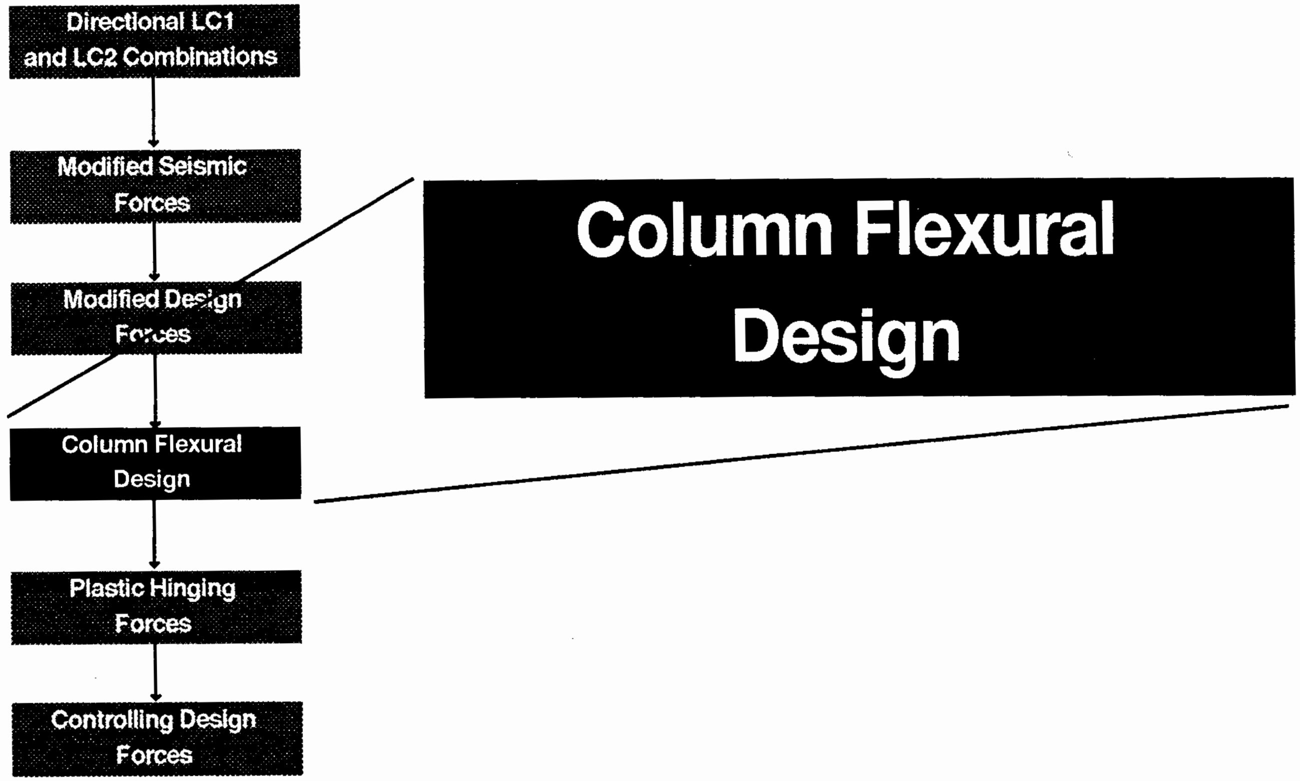
Column Design Forces / Flexure

	Per AASHTO	Elastic R=1	Modified R=5
Axial	Use Full Elastic Value	1050 kip 1146 kip	Not Used
Moment	Use Modified Value	Not Used	2332 kip ft

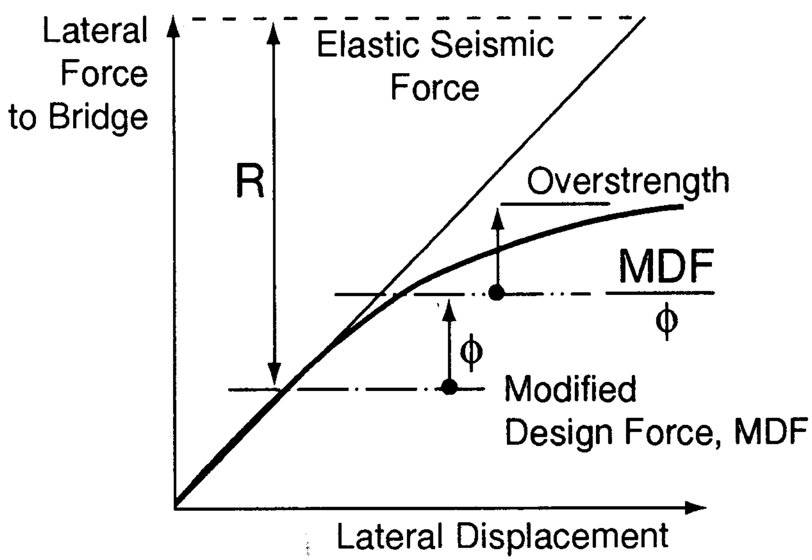
LC1 Controls

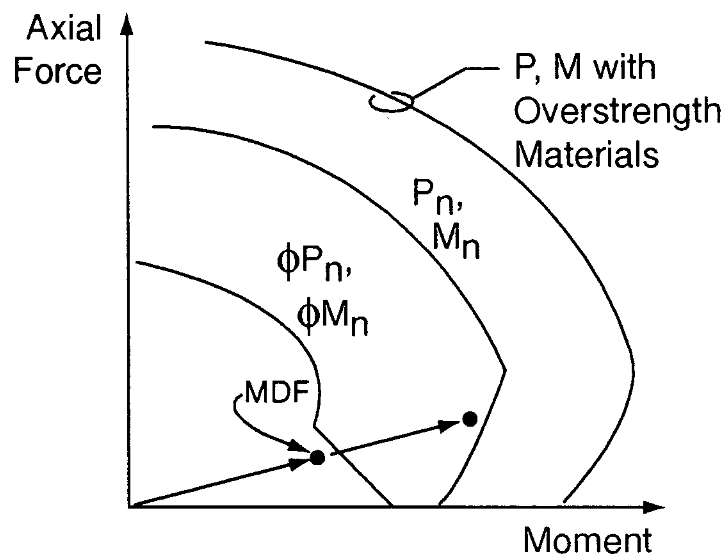
Longitudinal Load Case (Primary)

From Elastic Seismic Forces To Design Forces



Realistic Forces and Internal Moments





Controlling Forces for Design

- Design Shear Resistance, Connections, Foundations, etc., for ‘Overstrength Forces’ (Plastic Hinging Forces)

or

- Design for Elastic Forces

Whichever Is Smaller

Handling Material "Overstrength"

- **Realistic Values**

$f'_c \approx 1.5 \bullet (\text{nominal } f'_c)$ ...

Confinement
Conservative Mix Design
Strength Gain with Time

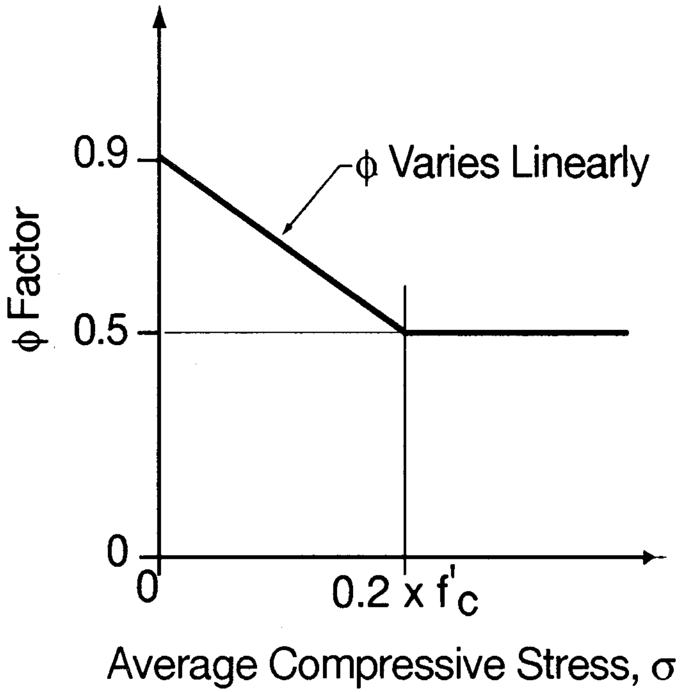
$f_y \approx 1.25 \bullet (\text{nominal } f_y)$ ...

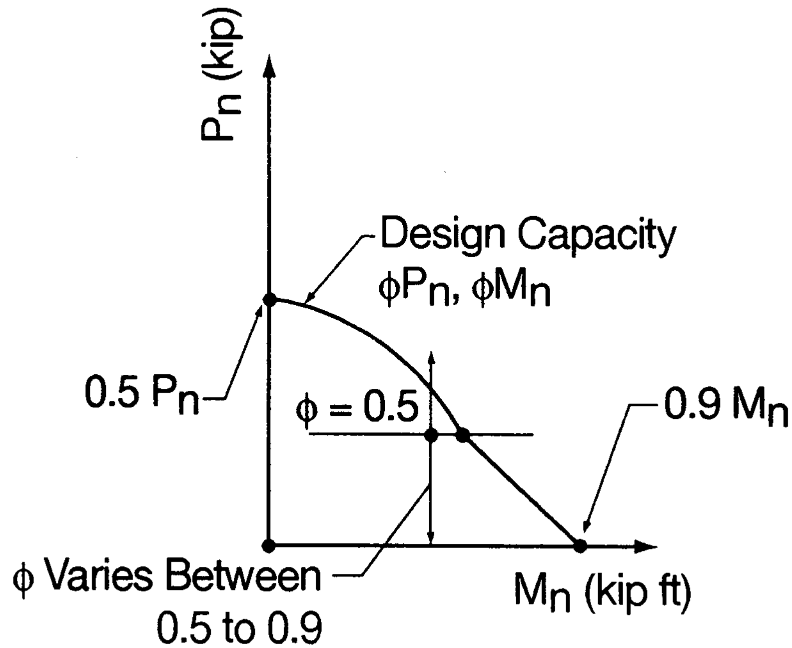
95% or More Bars Have
f_y Greater than Nominal
Strain Hardening

- **AASHTO Allows Simple 1.3 Increase in ϕ Factor**

$M_{actual} = 1.3\ M_n$

Strength Reduction Factor for SPC C and D



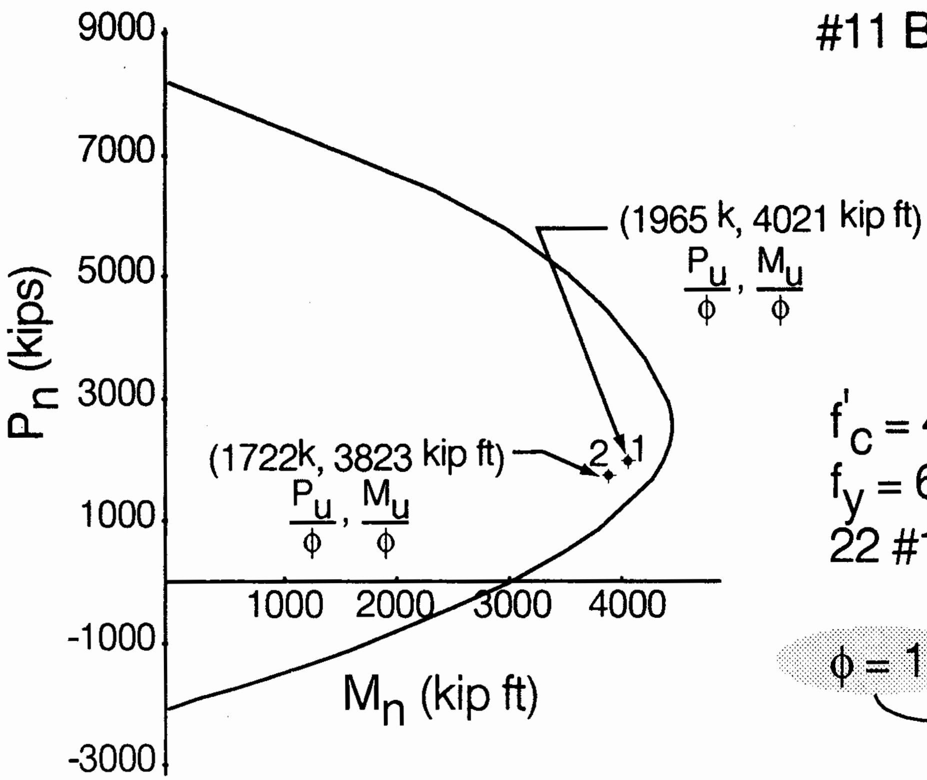


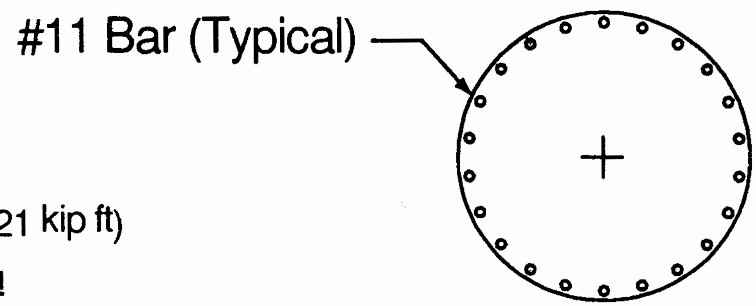
Required Strength of Column

	P (kips)	σ (psi)	$0.2\,f'_c$ (psi)	ϕ	$\frac{P}{\phi}$ (kip)	$\frac{M}{\phi}$ (kip ft)
Max	1146	633	< 800 → Interpolate ϕ	0.58	1965	4021
Min	1050	580		0.61	1722	3823

Column Diameter = 4 ft

Select Longitudinal Reinforcement



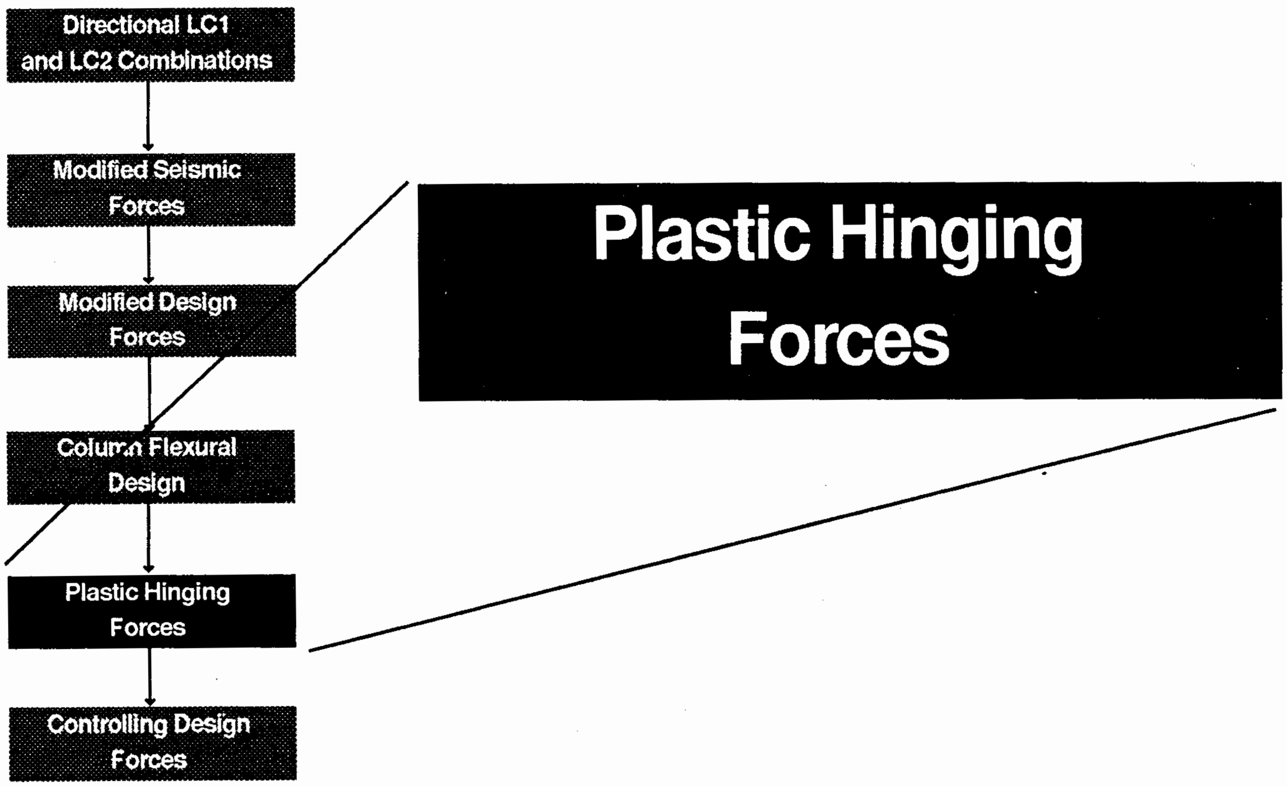


$f'_c = 4.0$ ksi
$f_y = 60.0$ ksi
22 #11 bars → ρ = **1.90% OK**

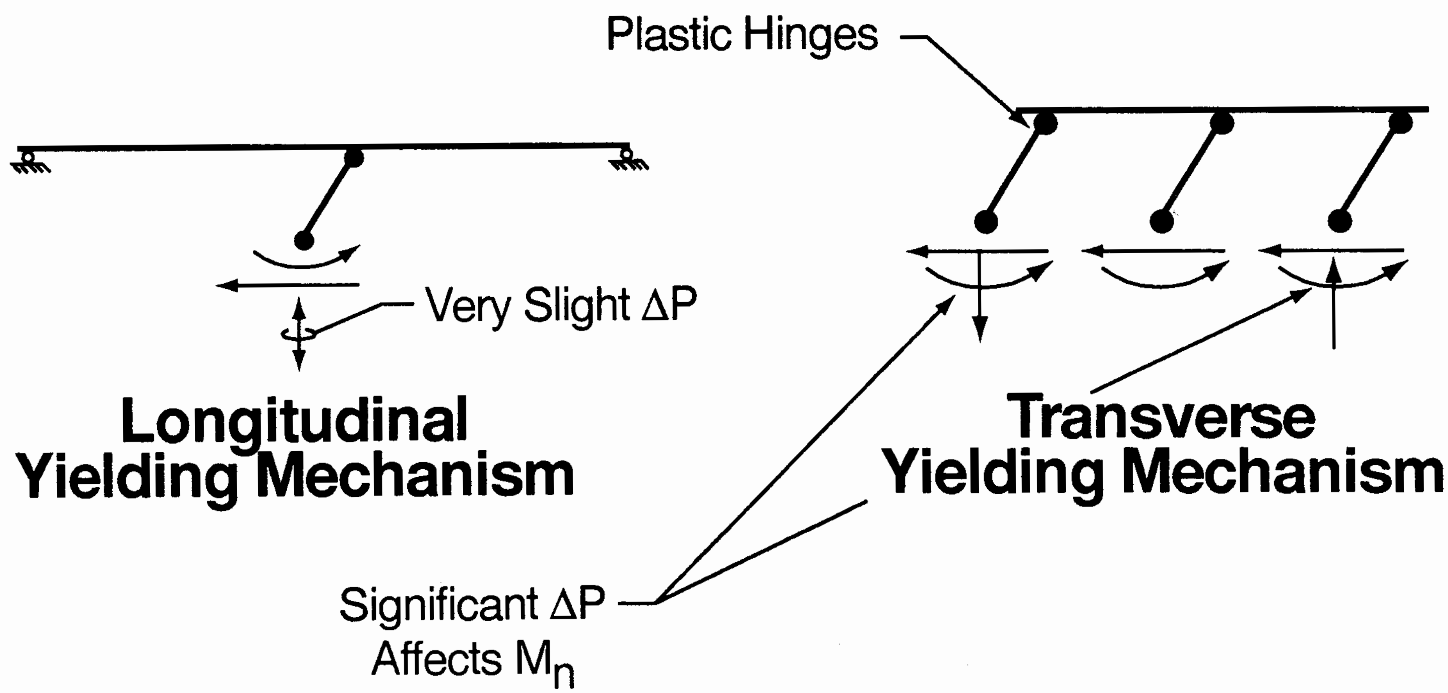
1% Minimum
6% Maximum

$\phi = 1.0$ — Nominal Strength Curve Plotted

From Elastic Seismic Forces to Design Forces



Plastic Hinging Behavior



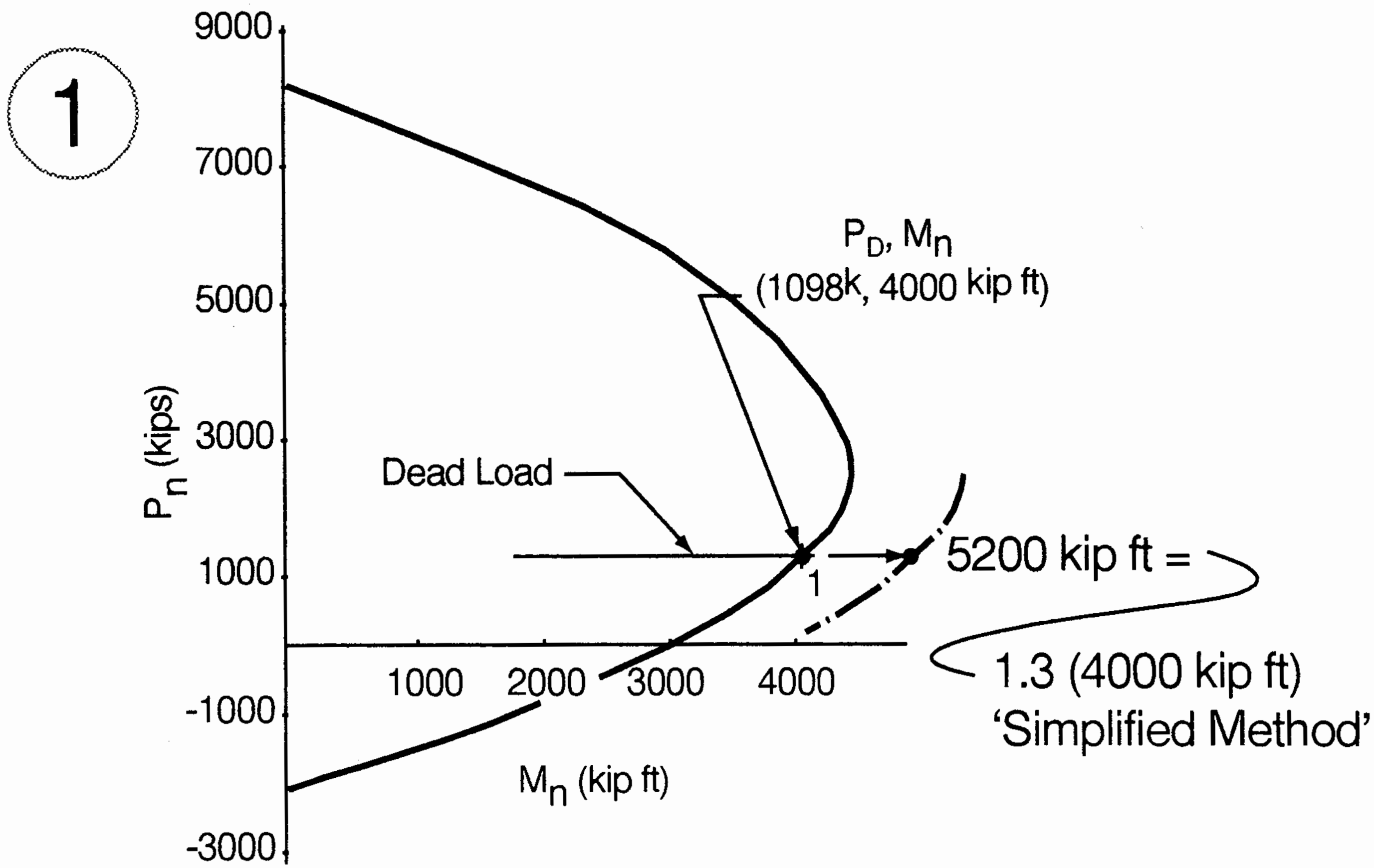
Outline / Plastic Hinging Forces / Multiple Column Bents

- **7.2.2 (B) (I-A)**

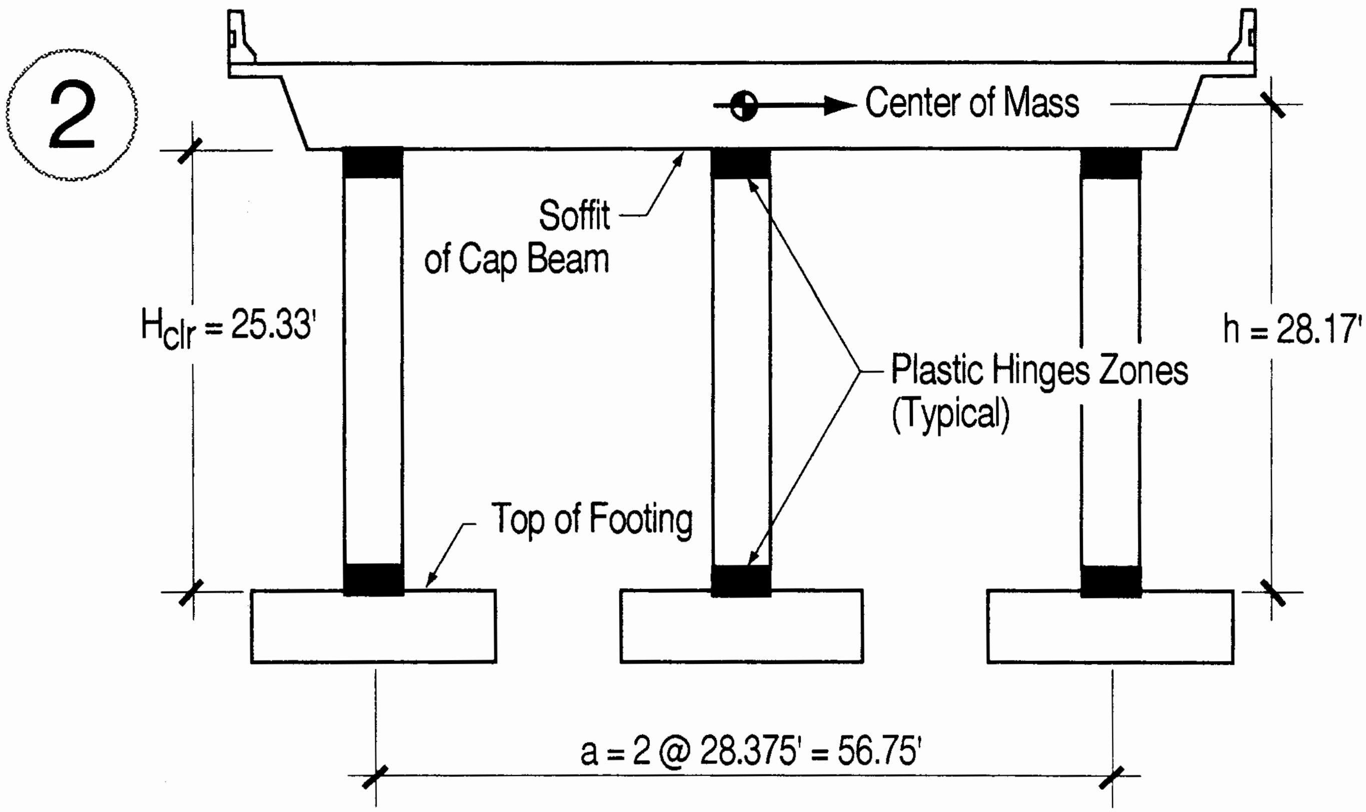
1. Determine M_p for Axial Dead Load, P_D
2. Calculate Column Shears, V
3. Apply Total Shear, ΣV, to Bent and Find ΔP
4. Determine Revised M_p and New Column Shears

Repeat if Axial Force Has Not Converged

Column Plastic Hinging Forces / Step 1

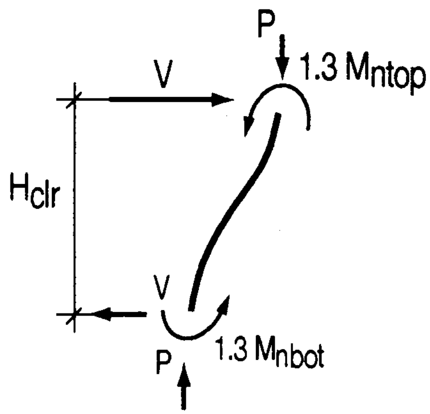


Transverse Plastic Hinging / Step 2



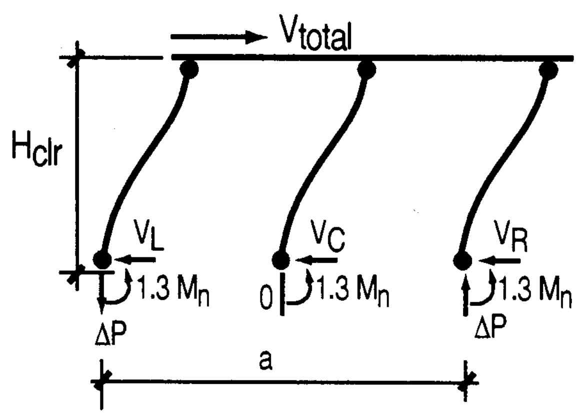
Elevation of Bent

Transverse Plastic Hinging (continued)



$$V = \frac{1.3\,M_{ntop} + 1.3\,M_{nbot}}{H_{clr}}$$

FBD of a Column
with Plastic Hinges

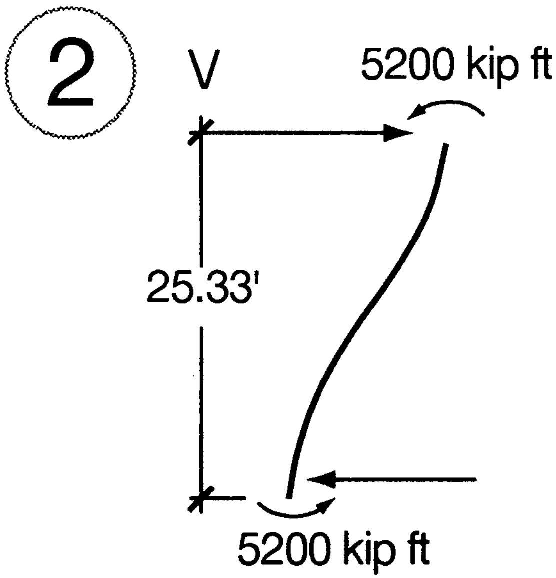


$$\Delta P = \frac{h\,(V_{total}) - \Sigma\,(1.3\,M_n)}{a}$$

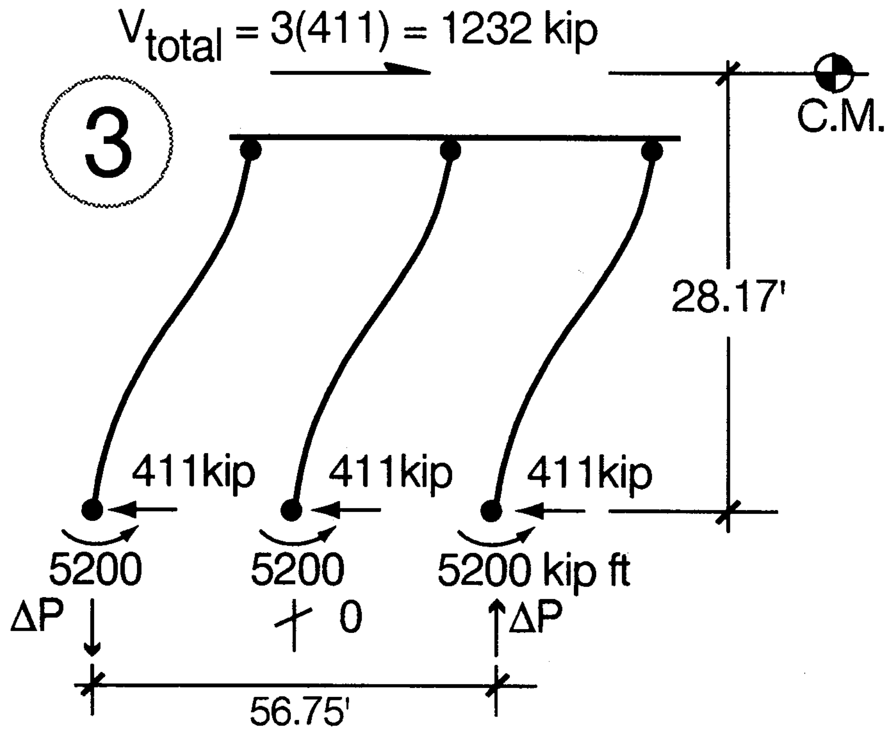
Seismic Forces on Columns when
Mechanism Has Formed

- Realistic V Depends on **Realistic** M_n ... Overstrength
- Realistic M_n Depends on **Realistic** Material Properties and ΔP

Column Plastic Hinging Forces / Steps 2 and 3



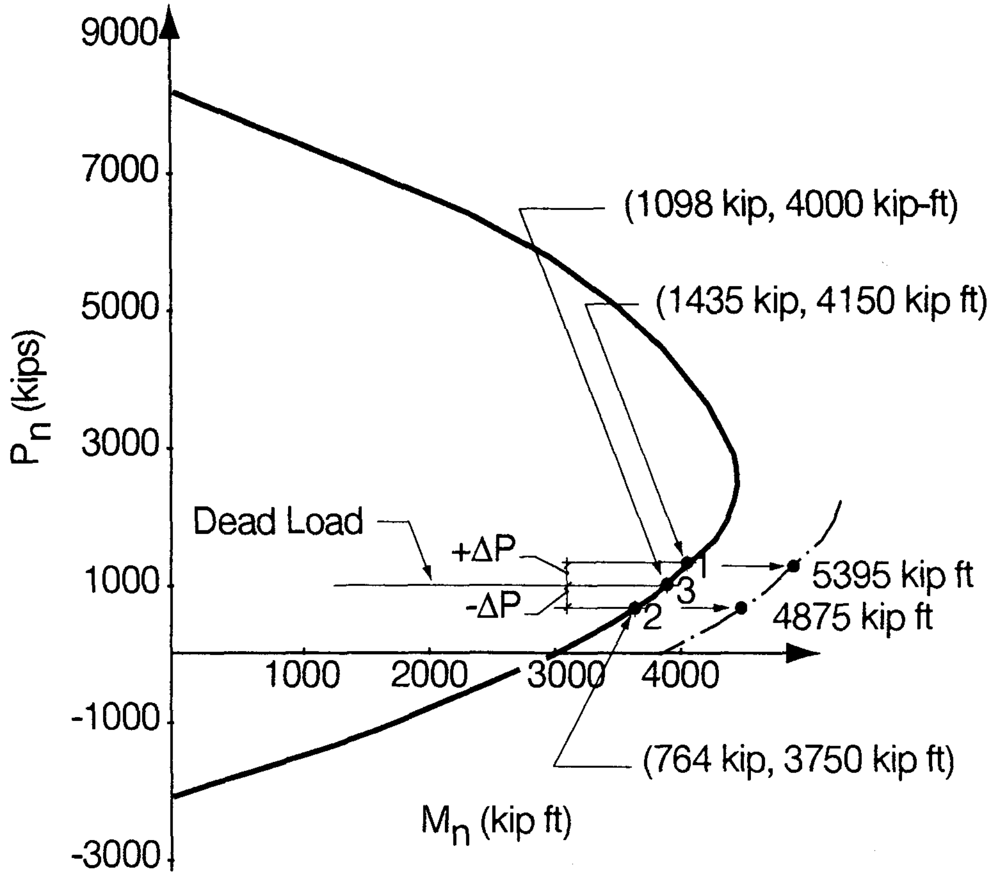
$$V_p = \frac{2(5200)}{25.33} = 411 \text{ kip}$$



$$\Delta P = \frac{1232\ (28.17) - 3(5200)}{56.75'} = 337 \text{ kip}$$

Column Plastic Hinging Forces / Step 4

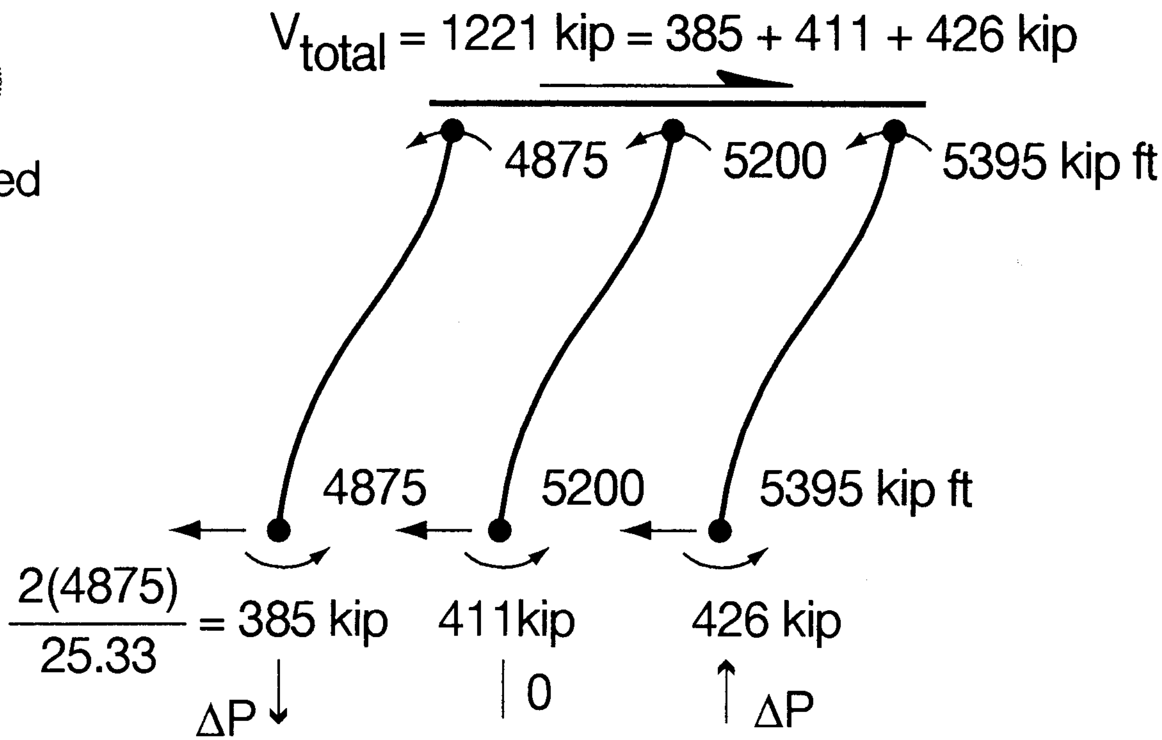
4



Plastic Hinging / Second Cycle

4

Continued



ΔP = 334 kip (vs. 337 kip Previous Value, Say OK)

Summary Plastic Hinging Forces

Transverse

Minimum Axial Load

$P_p = 764$ kip

$M_p = 4875$ kip ft

$V_p = 385$ kip

Maximum Axial Load

$P_p = 1432$ kip

$M_p = 5395$ kip ft

$V_p = 426$ kip

Longitudinal

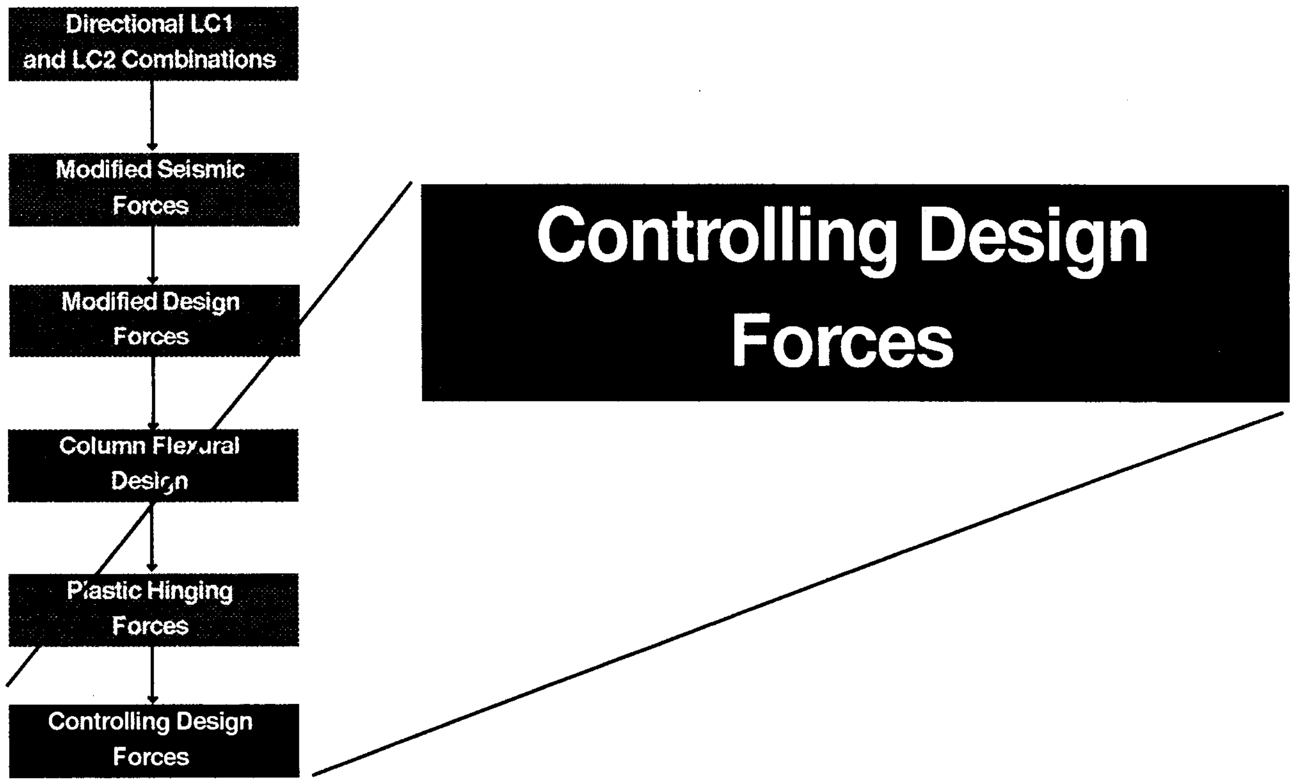
$P_p \sim 1098$ kip

$M_p \sim 5200$ kip ft

$V_p \sim 411$ kip

Bracketed by Transverse Value

From Elastic Seismic Forces to Design Forces



Column Design / Shear

	Elastic/Modified (R=1/Group Load) LC1	LC2	Plastic Hinging Long.	Trans. Max.	Trans. Min.
Shear	762 kip	248 kip (Resultants)	411 kip	426 kip	385 kip
Axial*	1146 kip 1050 kip	1152 kip 1044 kip	1098 kip	1432 kip	764 kip

Per AASHTO: Use Whichever Is Least

*Note: V_C Depends on Axial Load

Session 4

- Elastic Forces ⟶ Design Forces (Including Column Flexural Design)
- **Complete Column Design**
- Design Column Footings
- Abutment Issues

Column Design (SPC C and D)

Longitudinal Reinforcement — Already Done

↓

Confinement Reinforcement — End Regions / Hinging Zones; Typical / Middle Section

↓

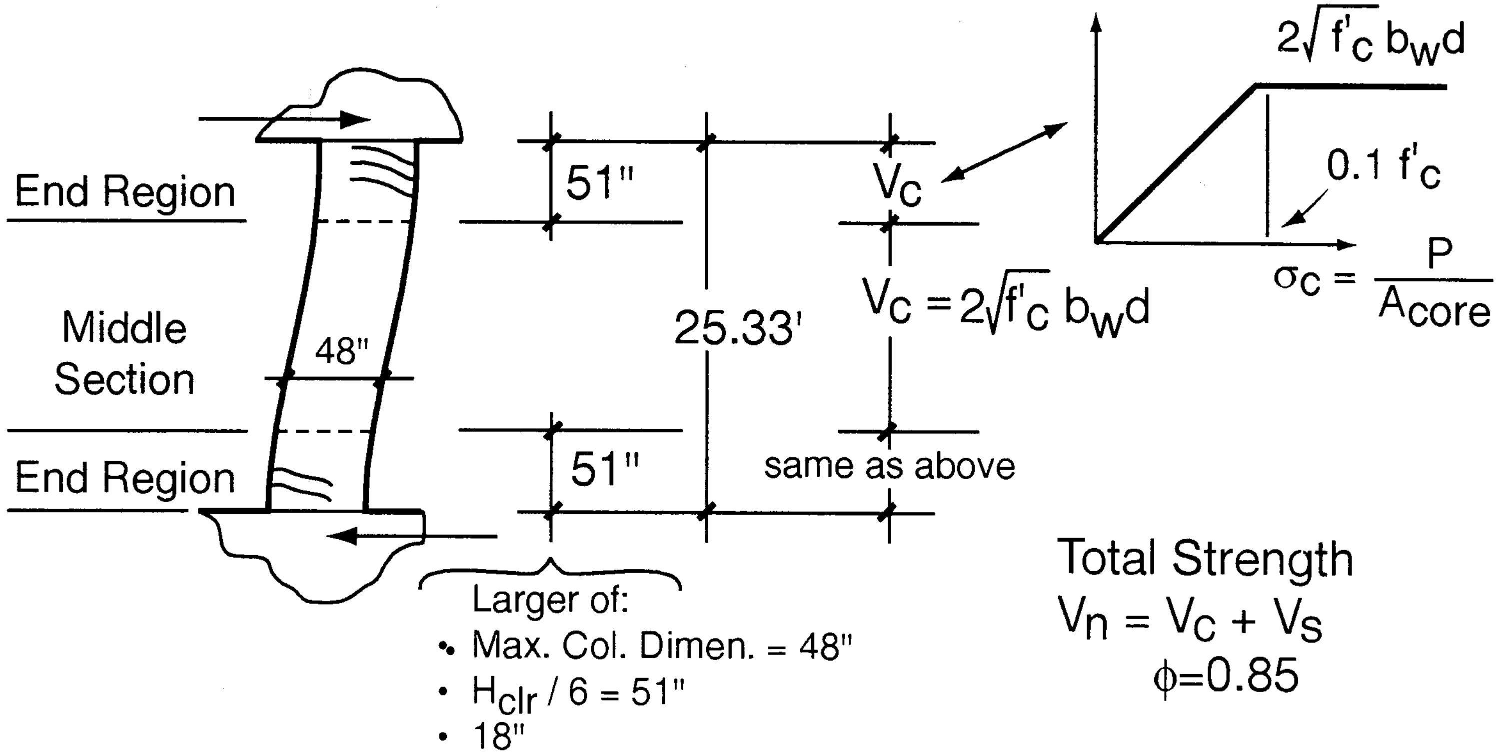
Shear Reinforcement — Quantity; Spacing

↓

Detailing — Development; Splices

Shear Strength

- **Strength – Two Zones:**



Total Strength

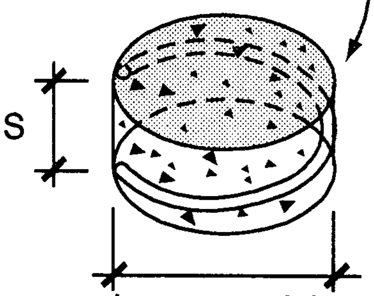
$V_n = V_c + V_s$

$\phi = 0.85$

Confined Plastic Hinge Zone

- **Spirals**

$$\rho_s = \frac{V_{steel}}{V_{core}}$$



$$\rho_s = 0.45\left(\frac{A_{gross}}{A_{core}} - 1\right)\ \frac{f'_c}{f_{yh}} = 0.0057$$

Minimum $\rho_s \geq 0.12\ \frac{f'_c}{f_{yh}} = 0.008$ Controls

Try s = 3.5" (s ≤ d_c/4 = 12" **and** s ≤ 4")

$$A_{sp} = \frac{\rho_s\ s\ d_{core}}{4d_s} = \frac{0.008\ (3.5)44^2}{4(44-0.625)} = 0.31\ \text{in}^2\ \text{Single Leg}$$

Use #5 Spiral at 3.5" Pitch for End Region

Shear Strength / End Region

- **End Region**

$$\frac{V_u}{\phi} = \begin{cases} \dfrac{426}{0.85} = 501 \text{ kip} & \text{Maximum Axial P} = 1432 \text{ kip} \\ \dfrac{385}{0.85} = 453 \text{ kip} & \text{Minimum Axial P} = 764 \text{ kip} \end{cases}$$

$$\sigma_c = \frac{P}{A_{core}} = \frac{764 \text{ kip}}{1521 \text{ in}^2} = \underbrace{0.50 \text{ ksi} > 0.40 \text{ ksi} = 0.1\, f'_c}_{V_c = 2\sqrt{f'_c}\, b_w d}$$

V_c - Same for Either Axial Load

Use 501 kip as Required Shear Strength, $\dfrac{V_u}{\phi}$

Shear Strength / End Region (continued)

$$V_c = \frac{2\sqrt{4000}}{1000}(48)(37.2) = 226 \text{ kip}$$

$$V_s = \frac{Vu}{\phi} - V_c = 501 - 226 = 275 \text{ kip}$$

Use of Plastic Hinging Shear
Prevents Brittle Shear Failure

For #5 Confinement Spiral at 3.5" Pitch $V_s = 395$ kip OK

Shear Strength / Middle Section

V_c = 226 kip (Same as End Region)

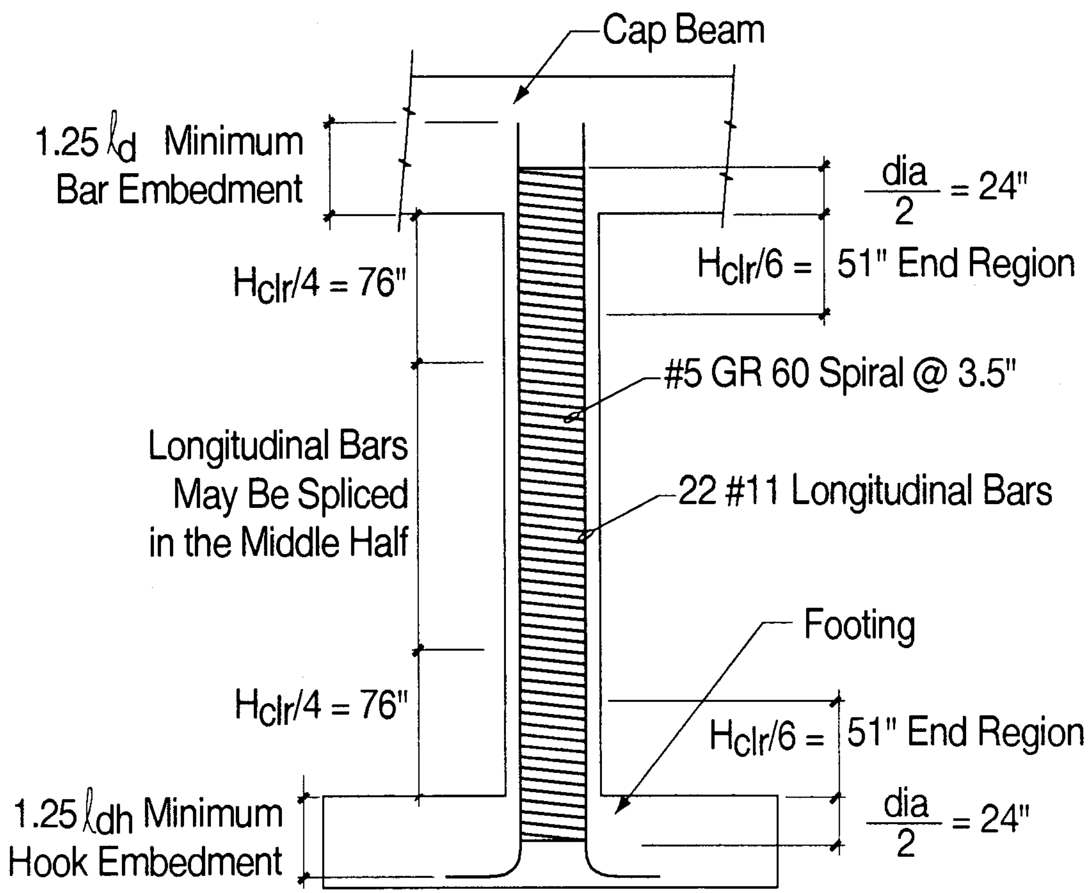
Required V_s = 275 kip (Same as End Region)

#5 Spiral @ 3.5" V_s = 395 kip > 275 kip

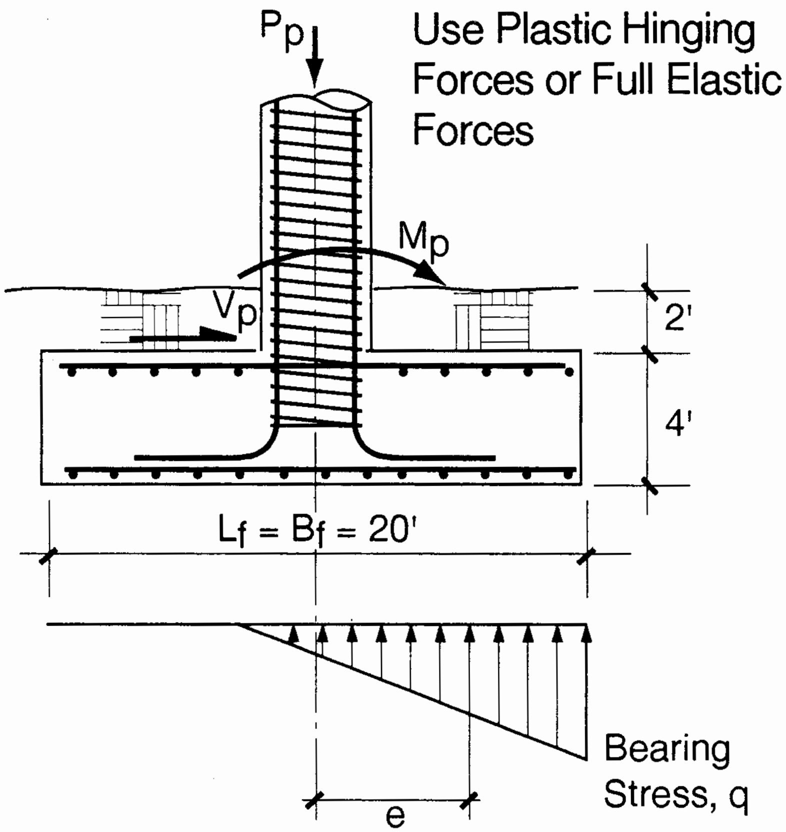
Could Open Up Spiral to 5" Pitch,
but Keep at 3.5" to Avoid Construction Errors

Use #5 @ 3.5" Pitch throughout Column

Column Reinforcement Details



Spread Footings



Issues

- **Soil Bearing Capacity**
  (q_{ult} = 24 ksf)
- **Overturning**
  (Uplift Over 1/2 Footing Dimension Is Permitted)
- **Sliding**
  (μ = 0.5, Neglect Passive)
- **Flexure in Footing**
  (Bottom and Top Steel)
- **Shear in Footing**
  (Stirrups?)

Spread Footings (continued)

- **Use Plastic Hinging Forces**
  (Transverse Maximum and Minimum)

	Limit	Maximum P	Minimum P	OK?
Bearing, q Stress	24 ksf	9.8 ksf	8.8 ksf	✔
Overturning, e	6.7 ft	4.0 ft	5.9 ft	✔
			Controls	
Sliding, μ required	0.5	0.24	0.35	✔

Could Reduce Footing Size Until e = b/3

Session 4

- Elastic Forces ⟶ Design Forces
  (Including Column Flexural Design)
- Design Columns
- Design Column Footings
- **Abutment Issues**

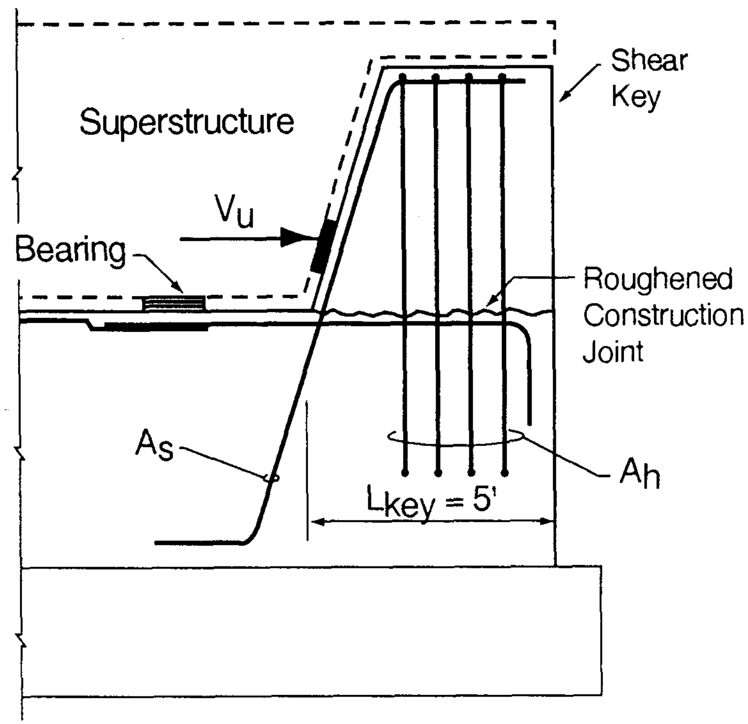
Abutment Shear Key

- **Use Shear Friction Design**

  $\mathbf{V_u = 1596\ kip}$ (based on R = 0.8)

  $\mathbf{L_{key} = 5\ ft}$ (based on $V_n < \begin{cases} 0.2\ f'_c \\ 800\ \text{psi} \end{cases}$

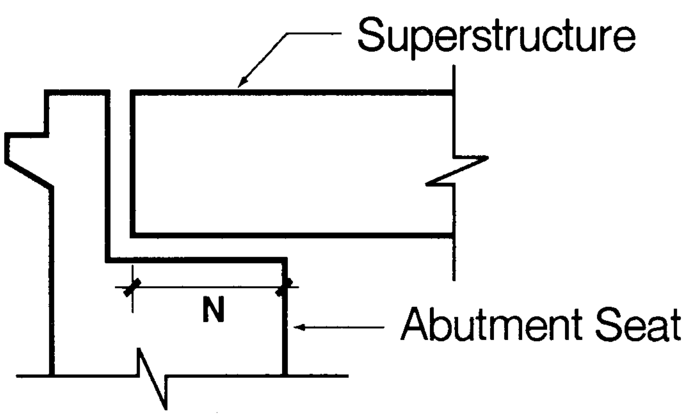
  $\mathbf{A_{vf} = 6.3\ in^2/ft\ of\ Length}$



Displacements and Seat Widths

For this Bridge Longitudinal Displacements Are Most Important

- Analysis — $\Delta_{elastic} = 0.24' \ (3'')$
- AASHTO — Seat Width, N, Prescriptive



$N > \Delta_{elastic}$

Observed Δ's Larger than Simple Analysis Indicates

Seat Width

- **For SPC C**

$$N = (12'' + 0.03L + 0.12H)(1+0.000125S^2)$$

L: Length of Unit **(feet)**

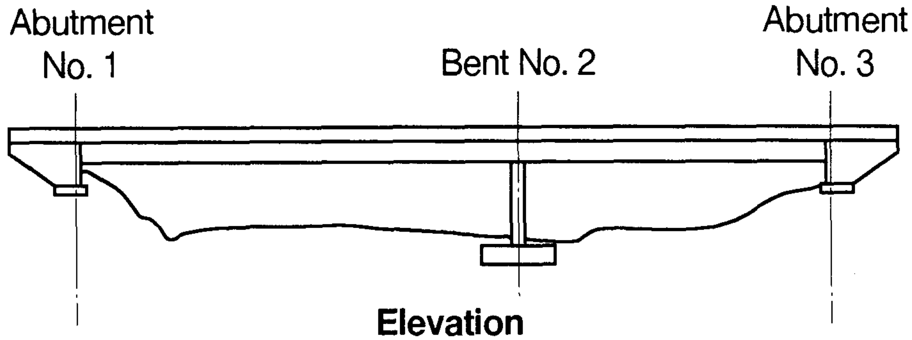
H: Average Height of Columns **(feet)**

S: Skew **(degrees)**

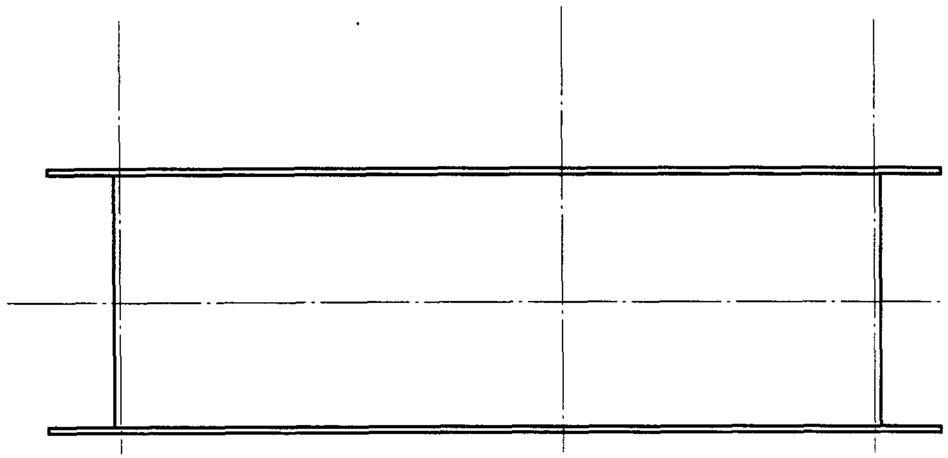
$$N = (12'' + 0.03(242') + 0.12(27.34'))(1+0.000125(0°)^2)$$

$$N = 22.5'' \ (1.88')$$

Summary of Example Design



Elevation



Plan

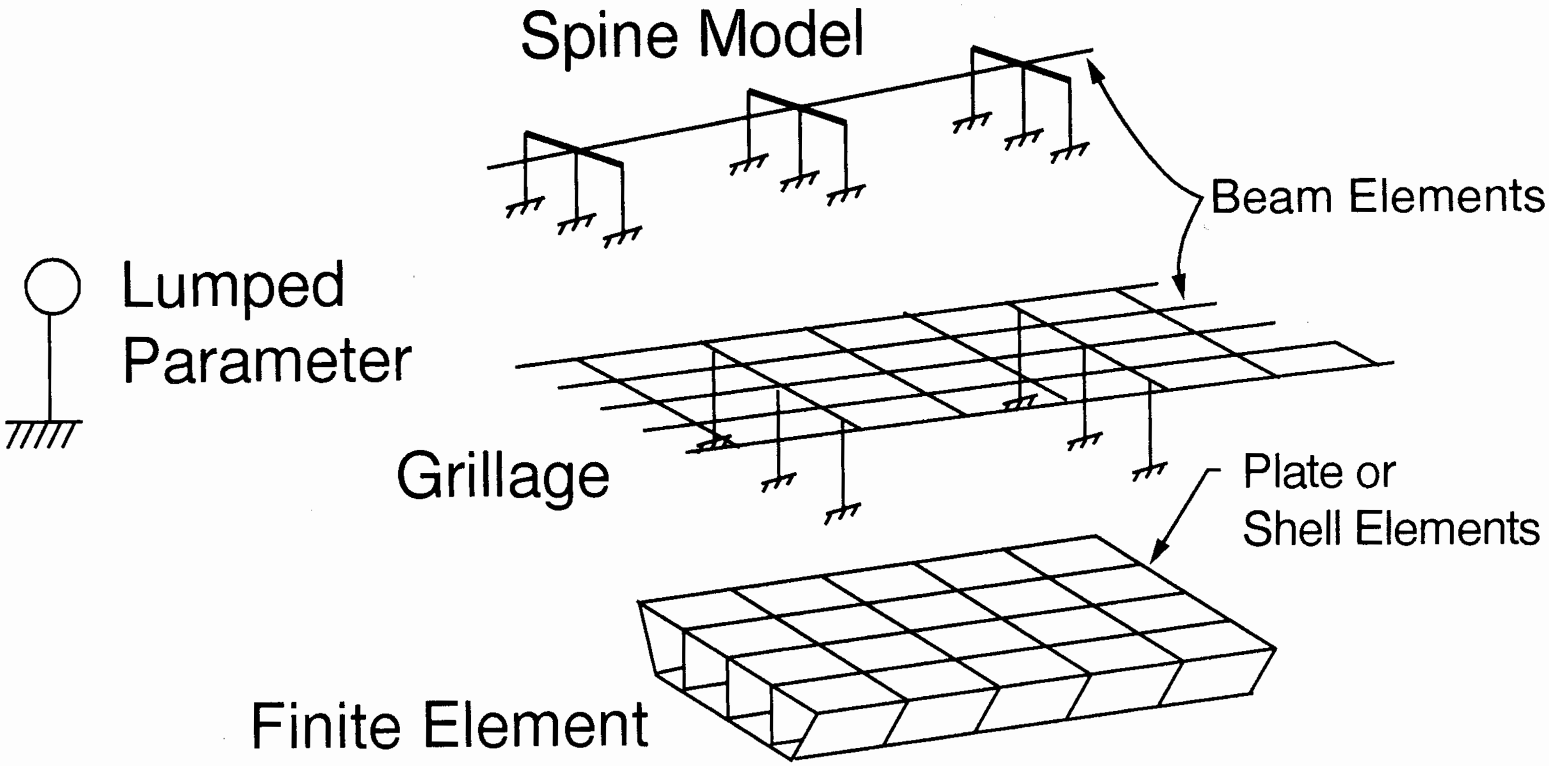
Session 5
Modeling Guidelines

- Types of Models
- Spine Model Considerations
- Properties
- Checking Modes

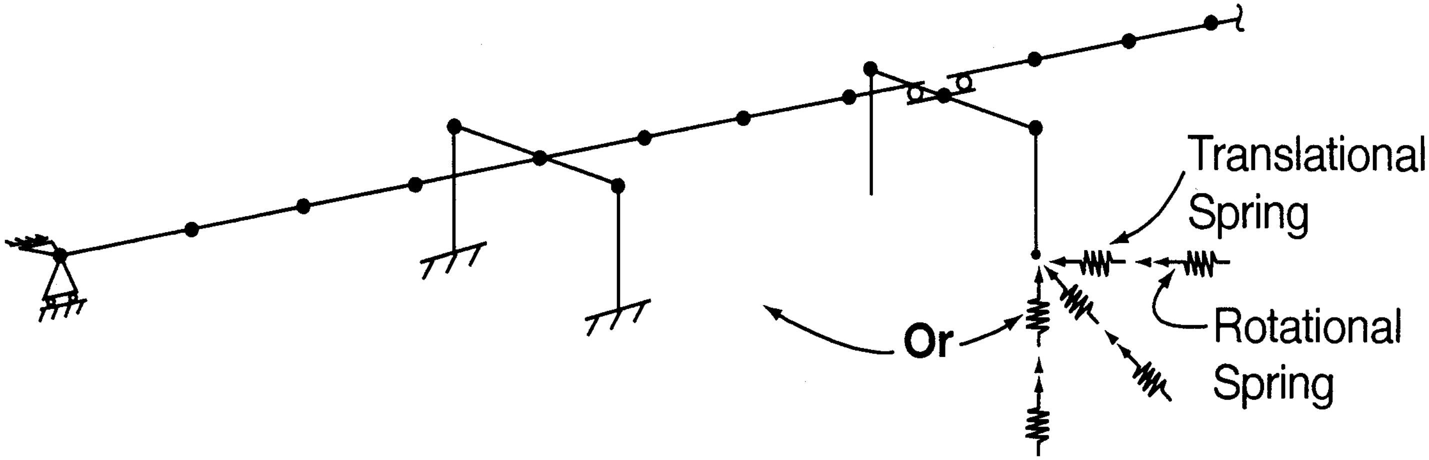
Modeling – General

- **Analytical Models Should Include:**
  Stiffness Distribution of Bridge
  Mass Distribution of Bridge (for Multimode Analysis)
- **Commonly 3D Models Are Used**
- **Standard Computer Programs Are Used for Analysis**

Types of Analysis Models

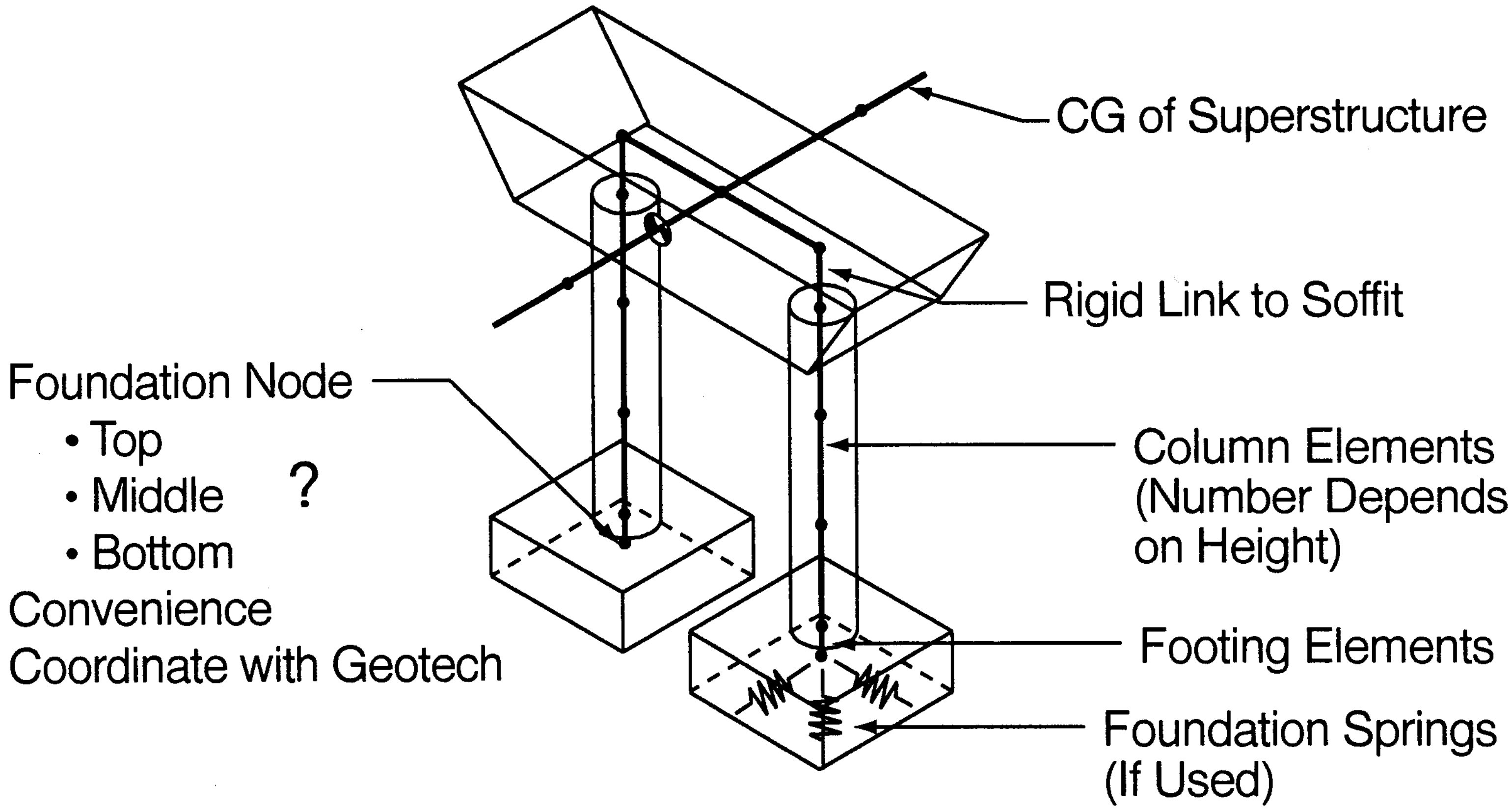


Spine Model for Seismic Analysis



- Substructure Elements Directly Modeled
- Superstructure Simple
- Include Connectivity Between Units
- Include Soil Springs / Releases / Fixity

Spine Model – Geometry Issues



Properties

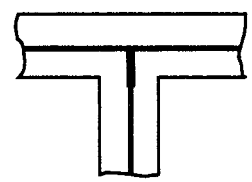
Stiffness
(e.g. Concrete)

E ~ $57\sqrt{f'_c}$ **Concrete**

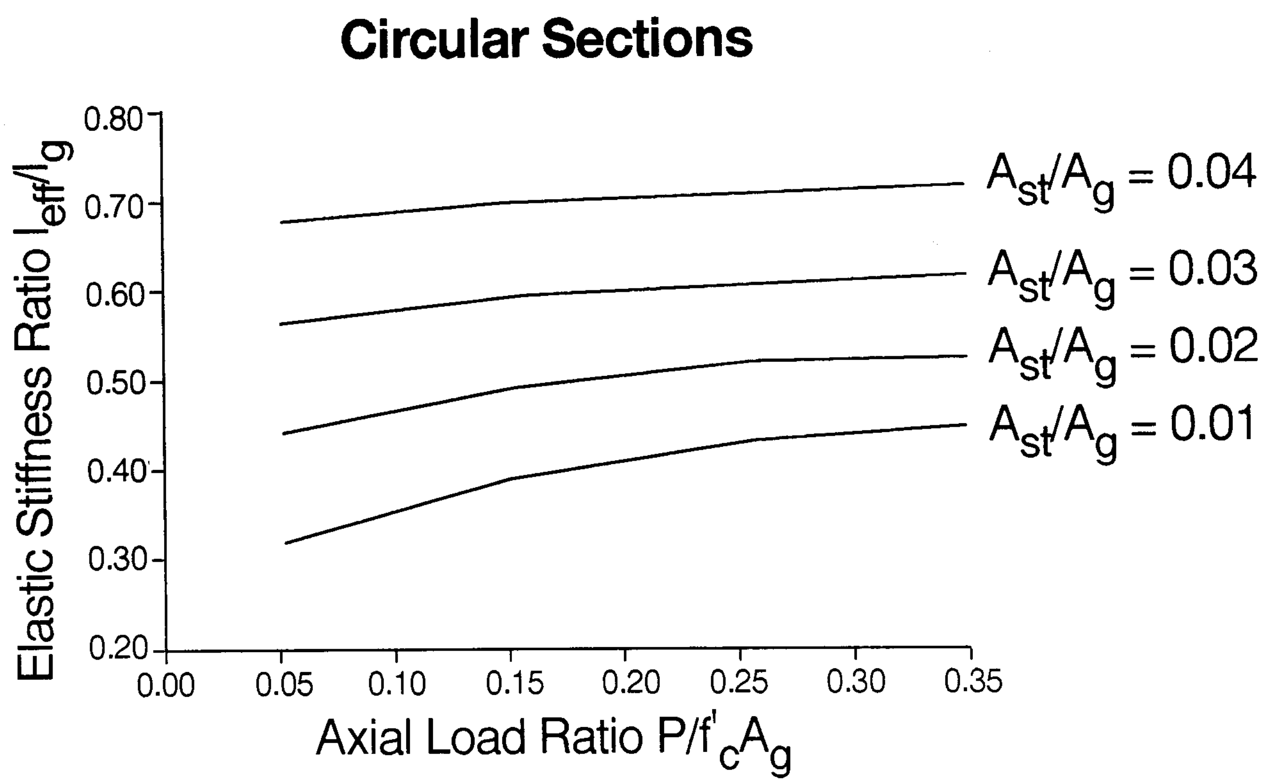
I ~ $\begin{cases} I_{gross} & \text{... Maximum Forces} \\ I_{effective} & \text{... Maximum Displacement} \\ I_{transformed} & \text{... Too High} \end{cases}$

A ~ **Area**

L ~ **Adjust for Joint Stiffness**



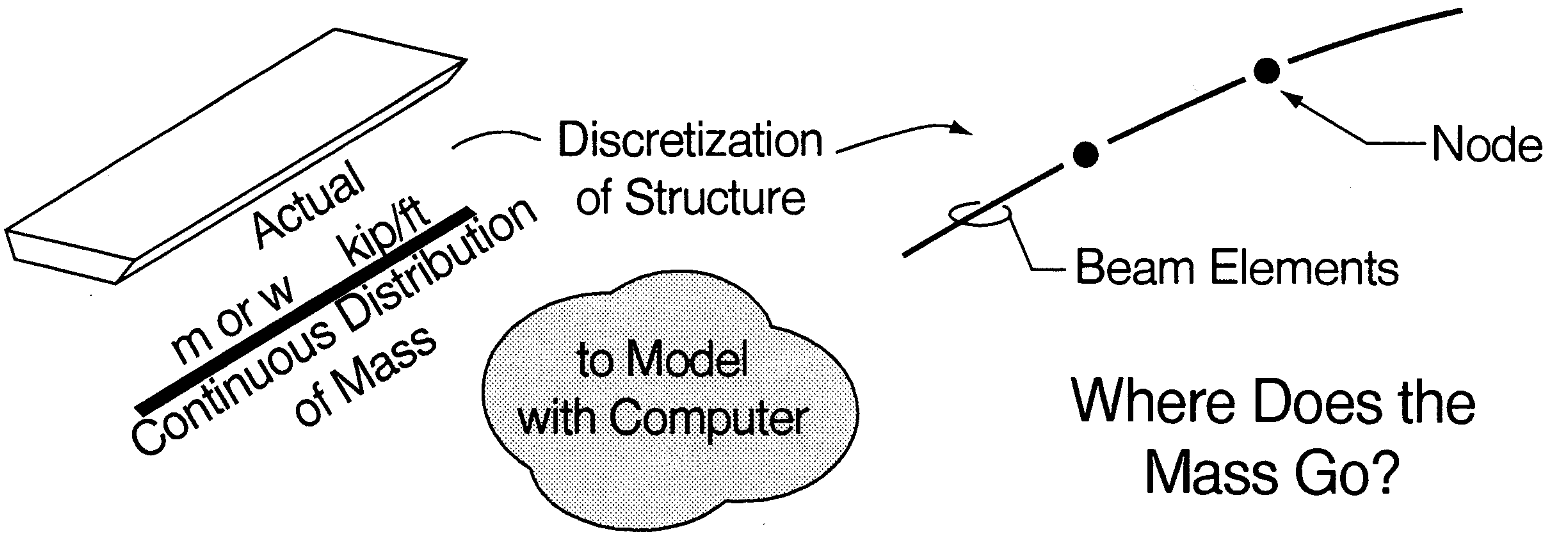
Effective Moment of Inertia – RC Columns



Priestley, Seible, and Calvi

Mass Distribution

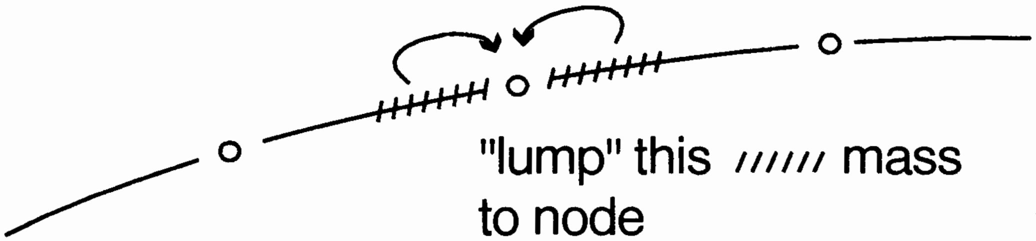
(or Weight $M = \frac{W}{g}$ ~~~ $F = Ma$)



Where Does the Mass Go?

Superstructure Mass Usually Most Significant

Mass Distribution (continued)



(example) Options:

members ⟶ • Specify density, σ, and area, A, of elemant

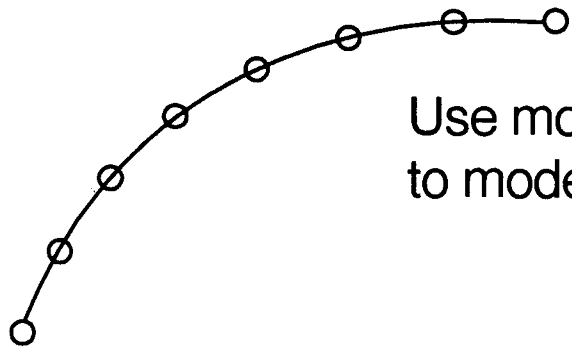
traffic barriers ⟶ • Specify weight per length, w

• Specify weight for node, W, directly

diaphragms ⟶

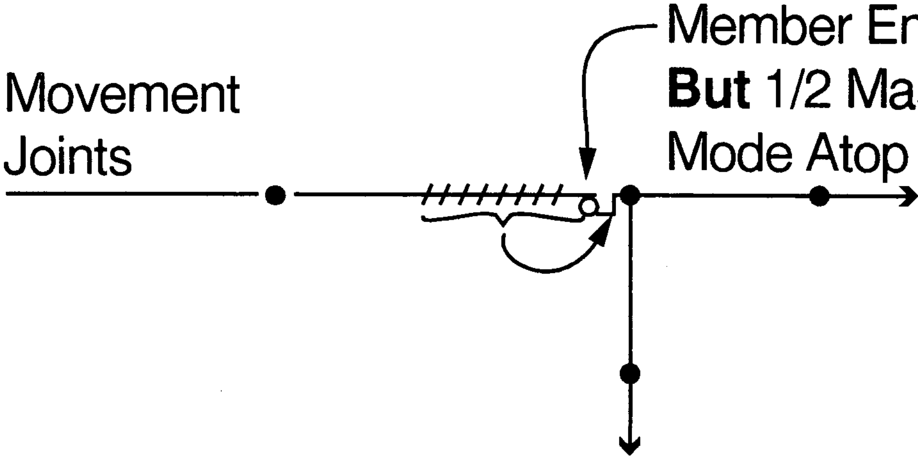
Special Considerations

Sharp Curves



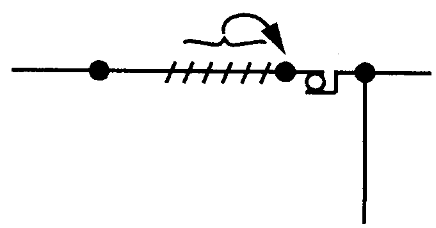
Use more elements to model curve

Movement Joints

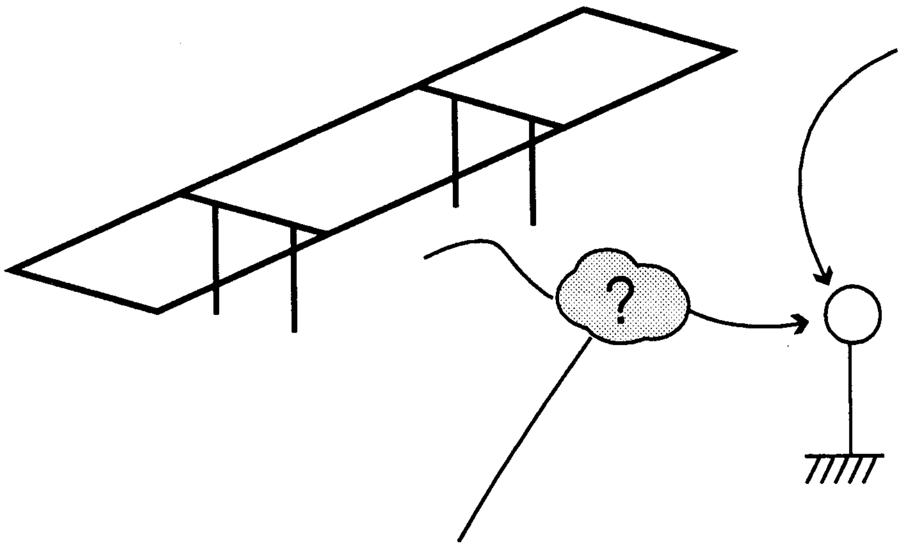


Member End Release Used **But** 1/2 Mass Assigned tto Mode Atop Column

Vs.



Lumped Parameter – Checking and Simple Cases



Single-Degree-of-Freedom System (SDOF)

W = Sum All Weights (or Prorate Weights)

K = Sum All Stiffnesses (or Prorate)

- Bents
- Abutments
- Superstructure

Reduction to SDOF System Requires a Good Estimate of the Displaced Shape

Session 5
Foundation Modeling

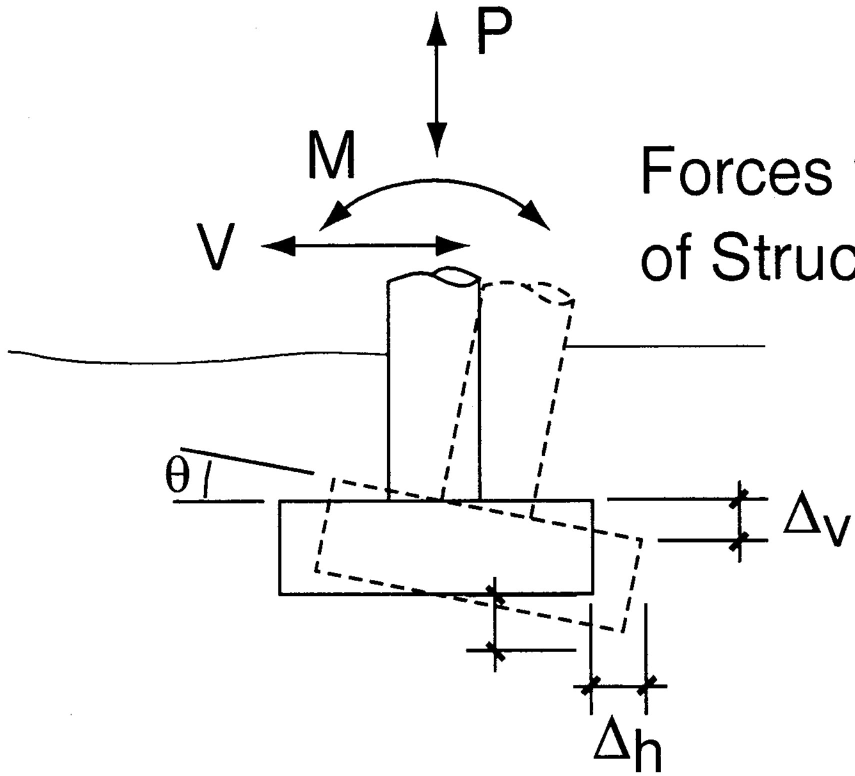
- **Structural / Geotech Relationship**
- **Soil Behavior**
- **Perspective — Using Soil Springs**
- **Modeling the Soil**

Foundation Analysis and Design Issues

- **This Session**
  General Behavior Concepts
  Simple Concepts for including Flexibility of Foundations
- **Next Seminar**
  More Detailed Analysis Techniques
  Discussion of Design Issues

Structural Engineer	Geotechnical Engineer
Needs	**Needs**
• Hazard / Spectra	• Substructure Types
• Foundation Concepts	• Soil Load Magnitudes
• Soil Properties	• Displacements
• Soil Capacities	• Comparison Types
• Modeling Assistance	Service
• Liquefaction Assessment	Ultimate

Foundation Behavior – Spread Footing

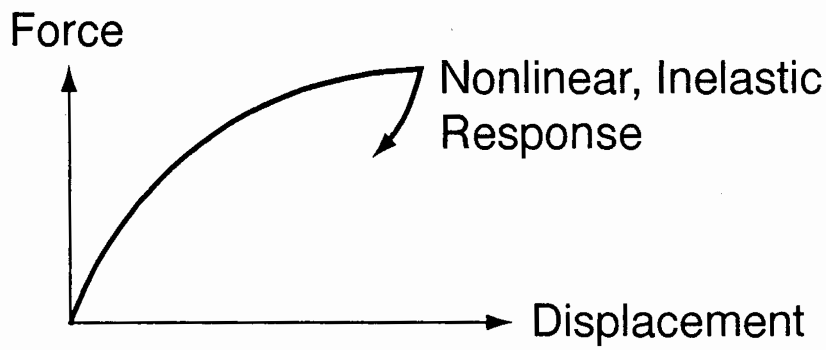


Forces from Movement of Structure Above

Flexibility of Supporting Soil

Foundation Behavior (continued)

Stiffness Effects



Damping Effects

Damping in Soil → Energy Dissipation (e.g. 'Radiation Damping')

Mass Effects

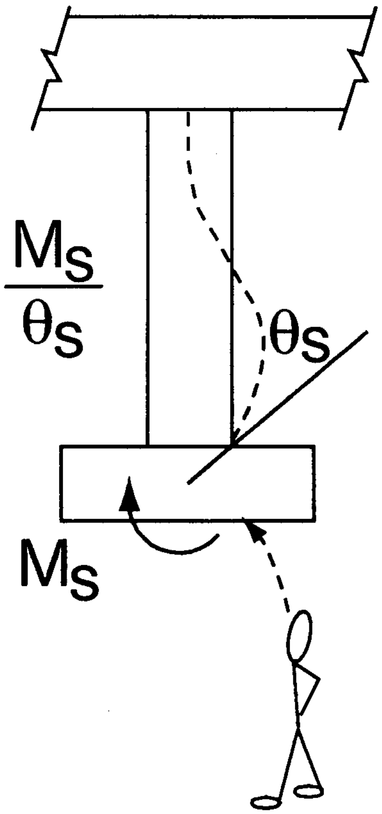
Soil Mass Affects Response

Using Spring Supports

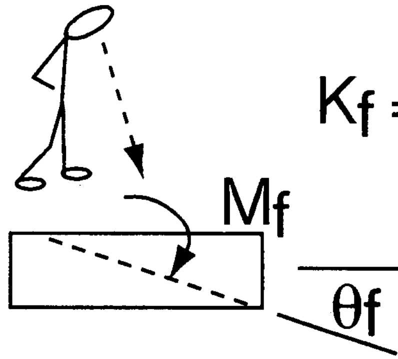
- When — **Refinement** of Seismic Analysis (**After** Bounding Analyses with Fixed or Free Supports)
- Why — Soil Flexibility Is Significant Relative to Structure
- How — Equivalent Linear Springs
- Accuracy — Actual Spring Constant Not as Important as Presence of Spring Itself

Stiffness – Structure vs. Foundation

$$K_s = \frac{M_s}{\theta_s}$$



$$K_f = \frac{M_f}{\theta_f}$$

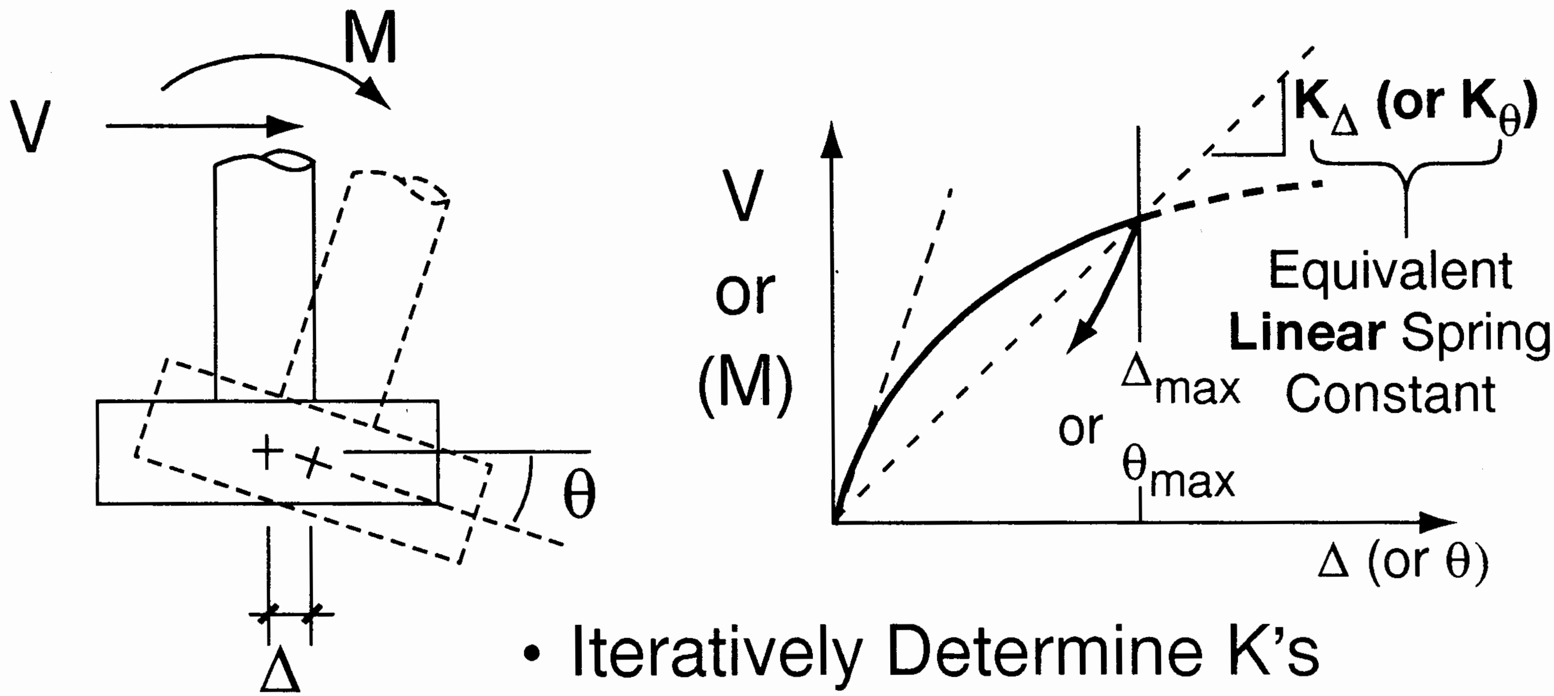


$K_f \gg K_s \longrightarrow$ Fixed

$K_f \ll K_s \longrightarrow$ Pinned

$K_f \approx K_s \longrightarrow$ Springs (or Bound)

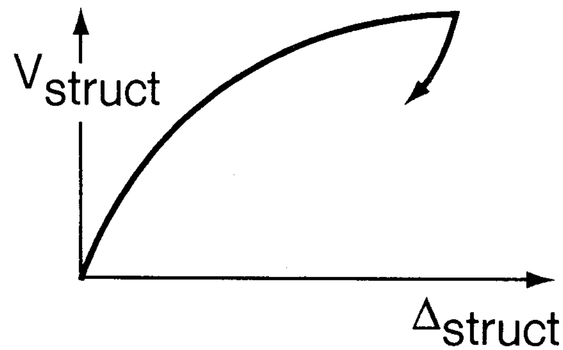
Soil Response May Be Nonlinear



- Iteratively Determine K's or 'Bound' Response

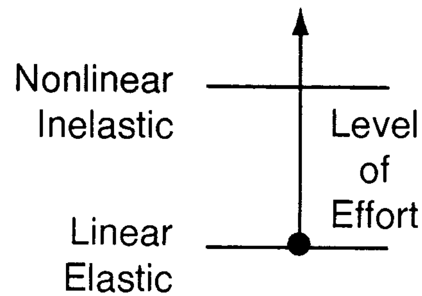
Perspective on Nonlinear Behavior

- Recall that Structure Nonlinear (Inelastic/Yielding) Behavior Is Expected

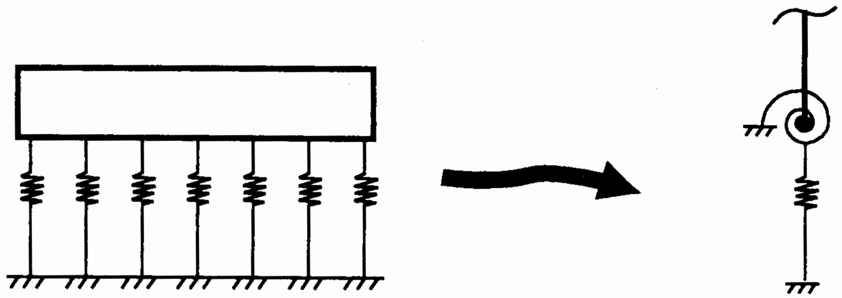


- Reasonable to Allow Some Nonlinear Soil Response

- Reasonable to Use **Elastic** Analysis



Modeling Soil Flexibility

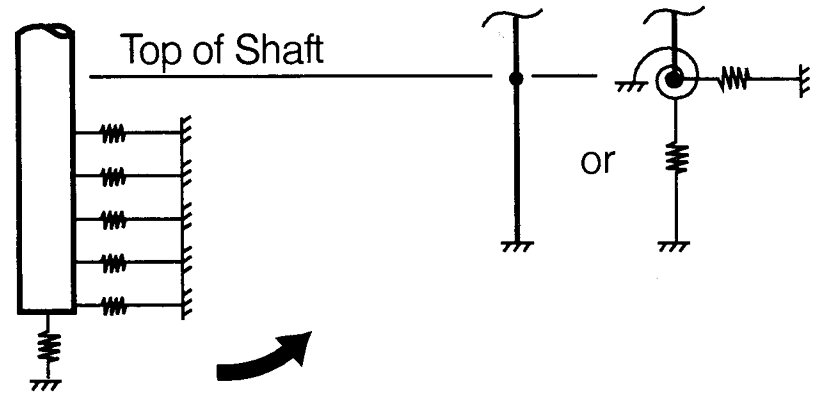
Foundation Type	Conceptual Model	Analytical Model
Spread Footing		

Reference: Bowles, 1988
FHWA - IP-87-6

Design Examples: 1, 2, 4

Modeling Soil Flexibility (continued)

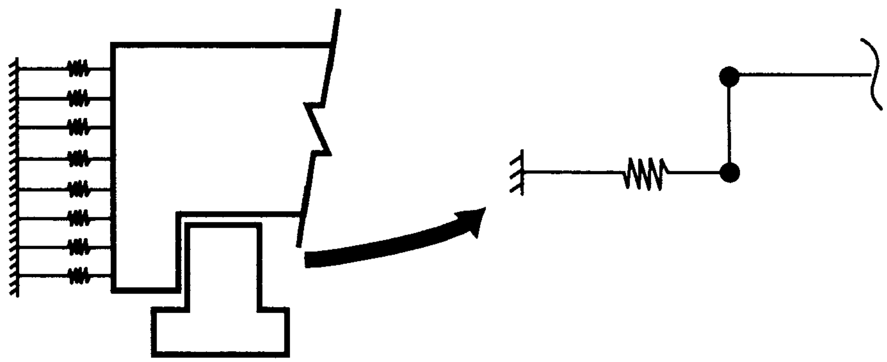
Foundation Types	Conceptual Model	Analytical Model
Piles **Drilled Shafts**		



Reference: NAVFAC DM 7.02, 1986
FHWA - IP-87-6

Design Examples: 5, 6

Modeling Soil Flexibility (continued)

	Conceptual Model	Analytical Model
Abutments/ Seat or Integral		

Reference: Caltrans 1995

Design Examples: 1, 3, 5, 6

Session 5
Multimode Dynamic Analysis

- **Definition**
- **Using Computational Tools**

  Input Data

  Process Flow

  Decisions

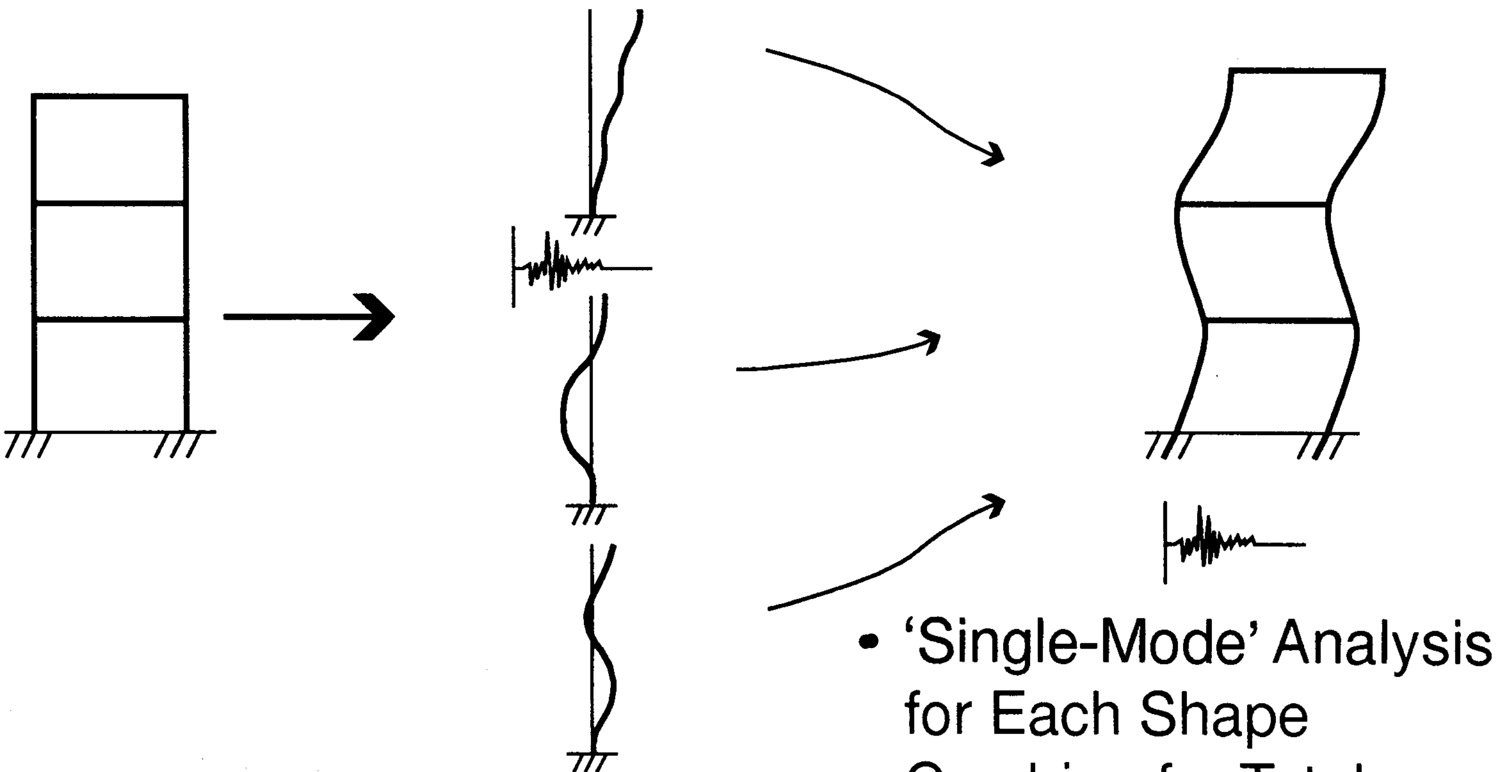
Multimodde Dynamic Analysis

- **What Is It?**

Superimpose Individual Mode Responses
to Estimate Structural Response

(Similar to Using Base Colors to Make Paint)

Multimode Concepts



- 'Single-Mode' Analysis for Each Shape
- Combine for Total

Multimode Dynamic Analysis

- **Why?**

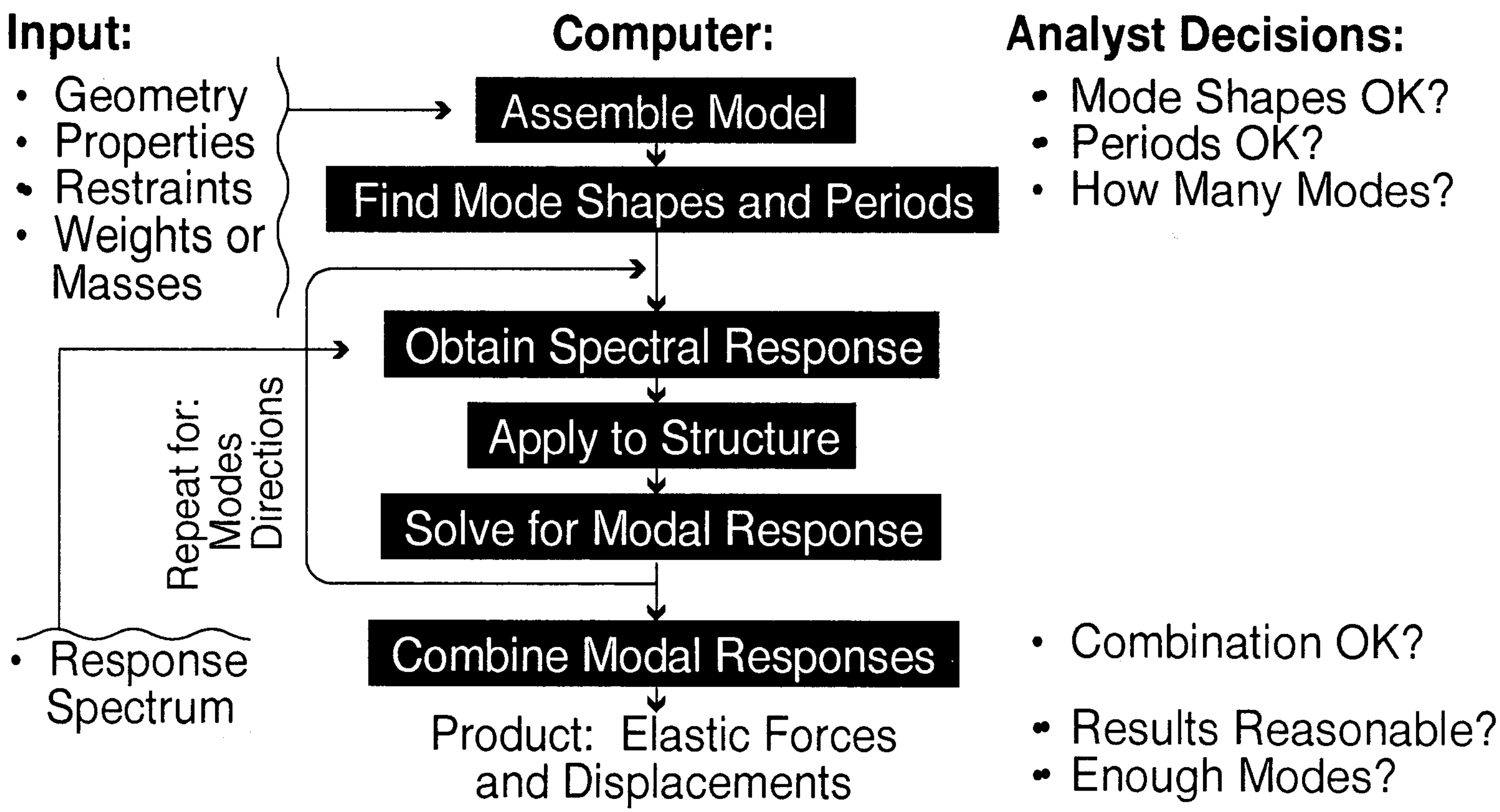
  Gives Reasonable Estimate of Forces and Displacements

  Especially Helpful for Complex and/or Irregular Structures

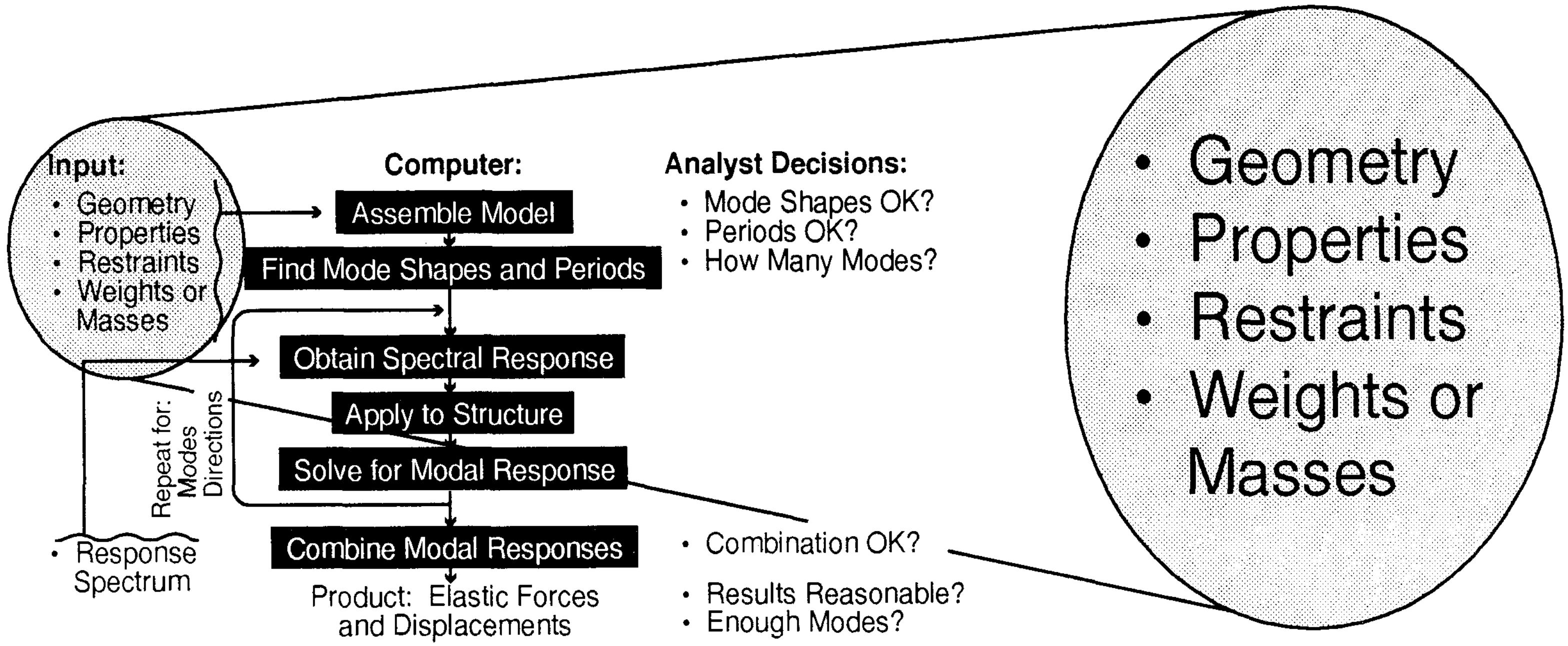
- **Limits?**

  Applies Only to Linear-Elastic (Non-Yielding) Structures

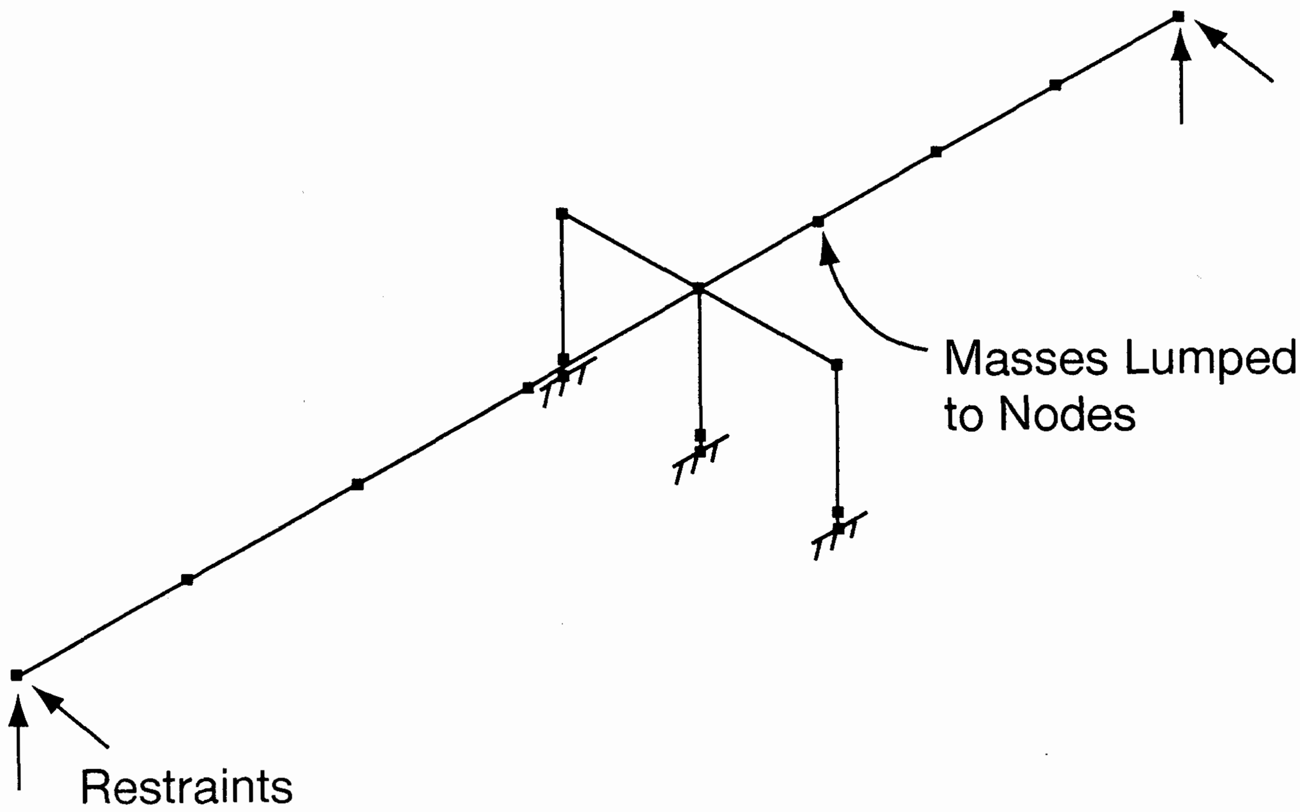
Multimode Dynamic Analysis



Multimode Dynamic Analysis



Example Bridge – Spine Model



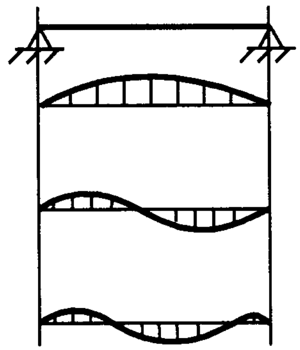
Mode Shape Terminology – 2D vs. 3D

2D

1st Mode ... Fundamental Mode

2nd Mode ... Higher Modes

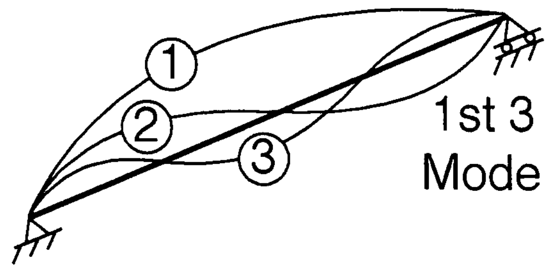
3rd Mode



3D ‘1st, 2nd, 3rd’ in Each Orthogonal Direction

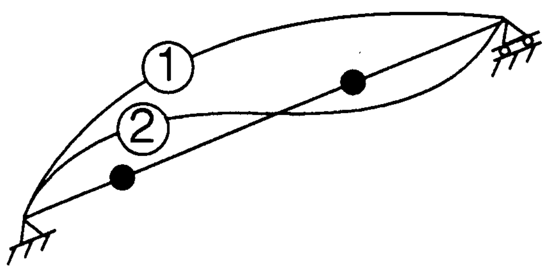
Breaking Structure into 'Discrete' Elements

Distributed Mass
Structure (Actual)

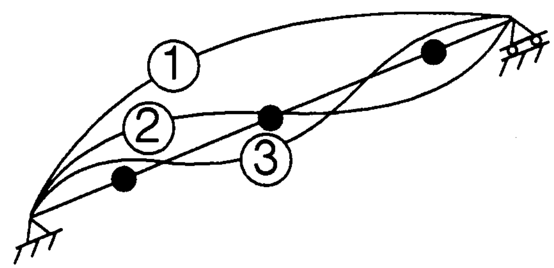


1st 3 Vertical
Mode Shapes

Two Nodes:
(2 Modes / Direction)



Three Nodes:
(3 Modes / Direction)

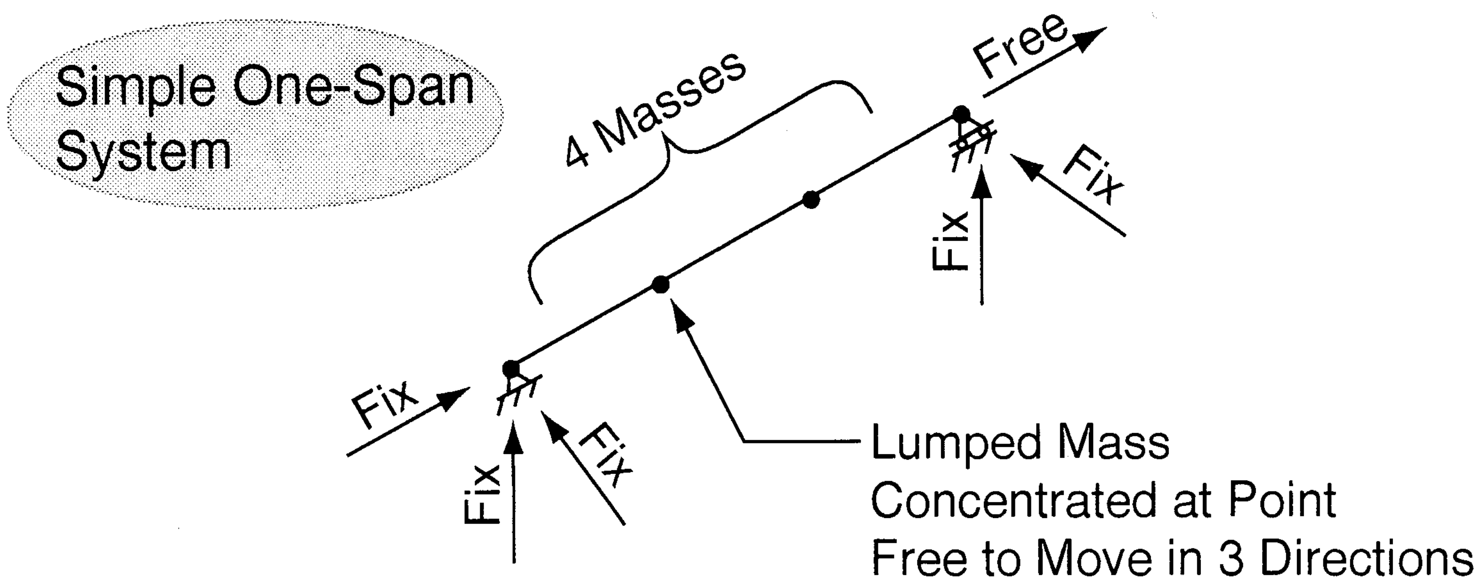


- More Nodes→Refinement of Forces
- AASHTO / Use 4 Elements (3 Nodes) per Span

Number of Modes Possible – 3D

Number of Modes Depends on:

- Number of Masses
- Boundary Conditions / Restraints

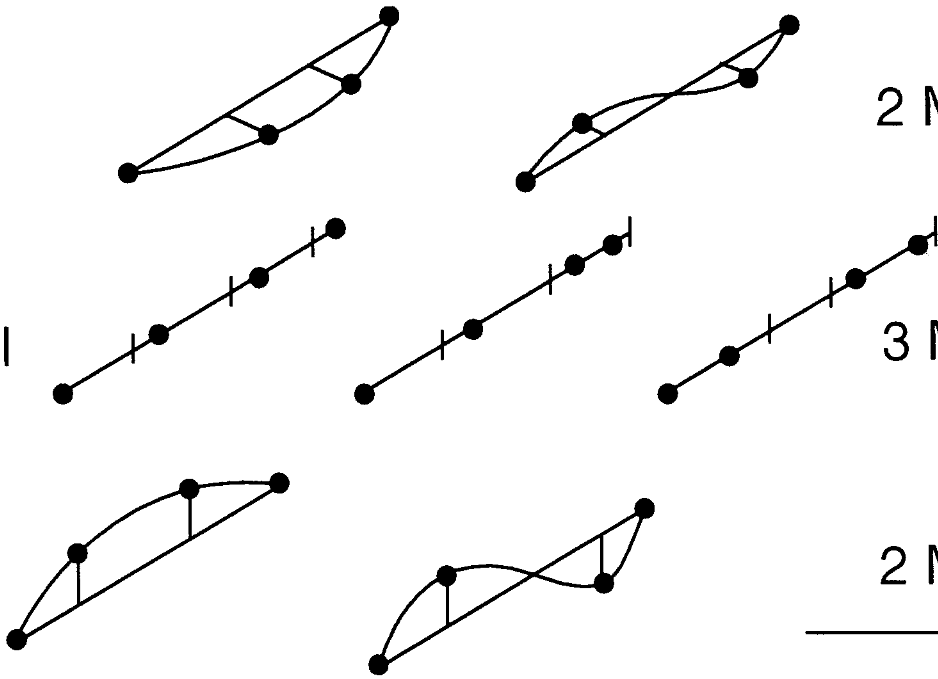


Number of Modes Possible (continued)

Transverse — 2 Modes

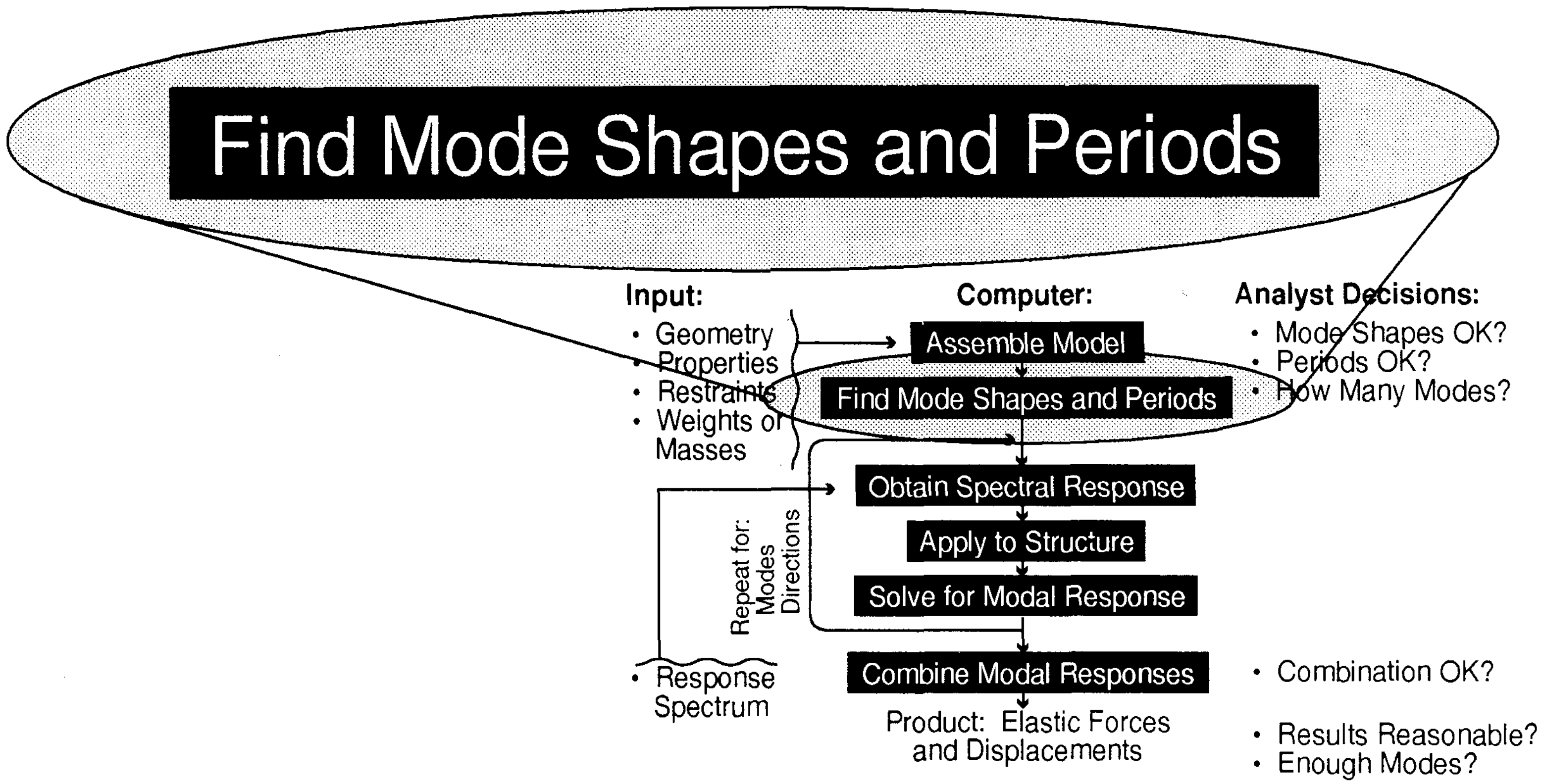
Longitudinal — 3 Modes

Vertical — 2 Modes



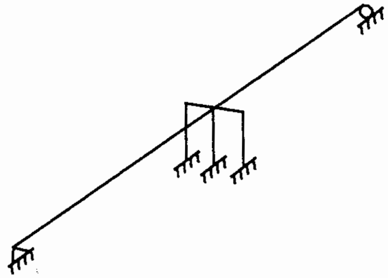
$$(3\underbrace{n_{masses}}_{4} - \underbrace{\text{Restraints}}_{5}) = 7 \text{ Modes}$$

Multimode Dynamic Analysis



Example Results – Ordering of Modes

E I G E N   S Y S T E M   P A R A M E T E R S



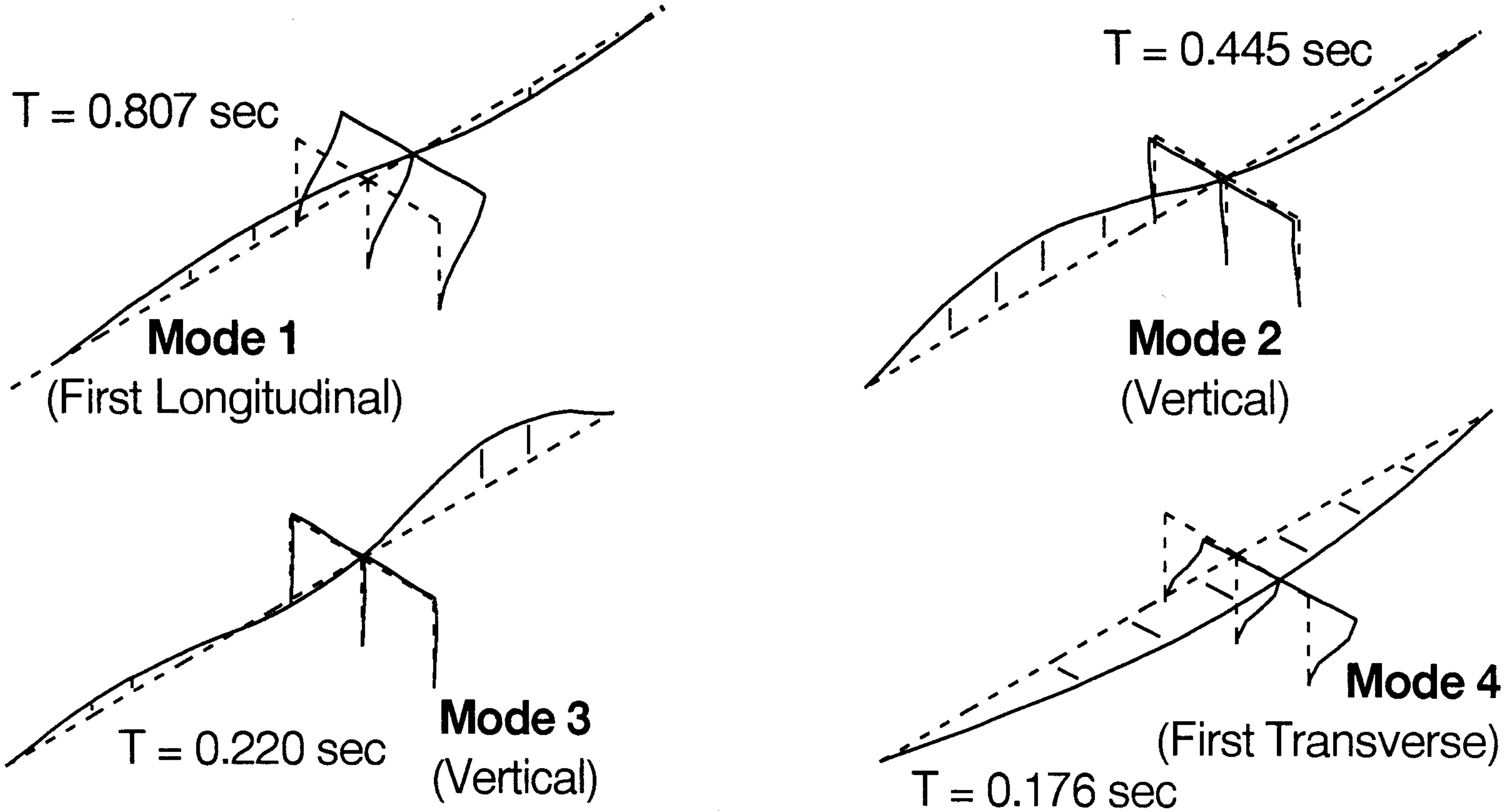
NUMBER OF EQUATIONS	=	78
NUMBER OF MASSES	=	38
NUMBER OF VALUES TO BE EVALUATED	=	15
SIZE OF SUBSPACE	=	19

E I G E N V A L U E S   A N D   F R E Q U E N C I E S

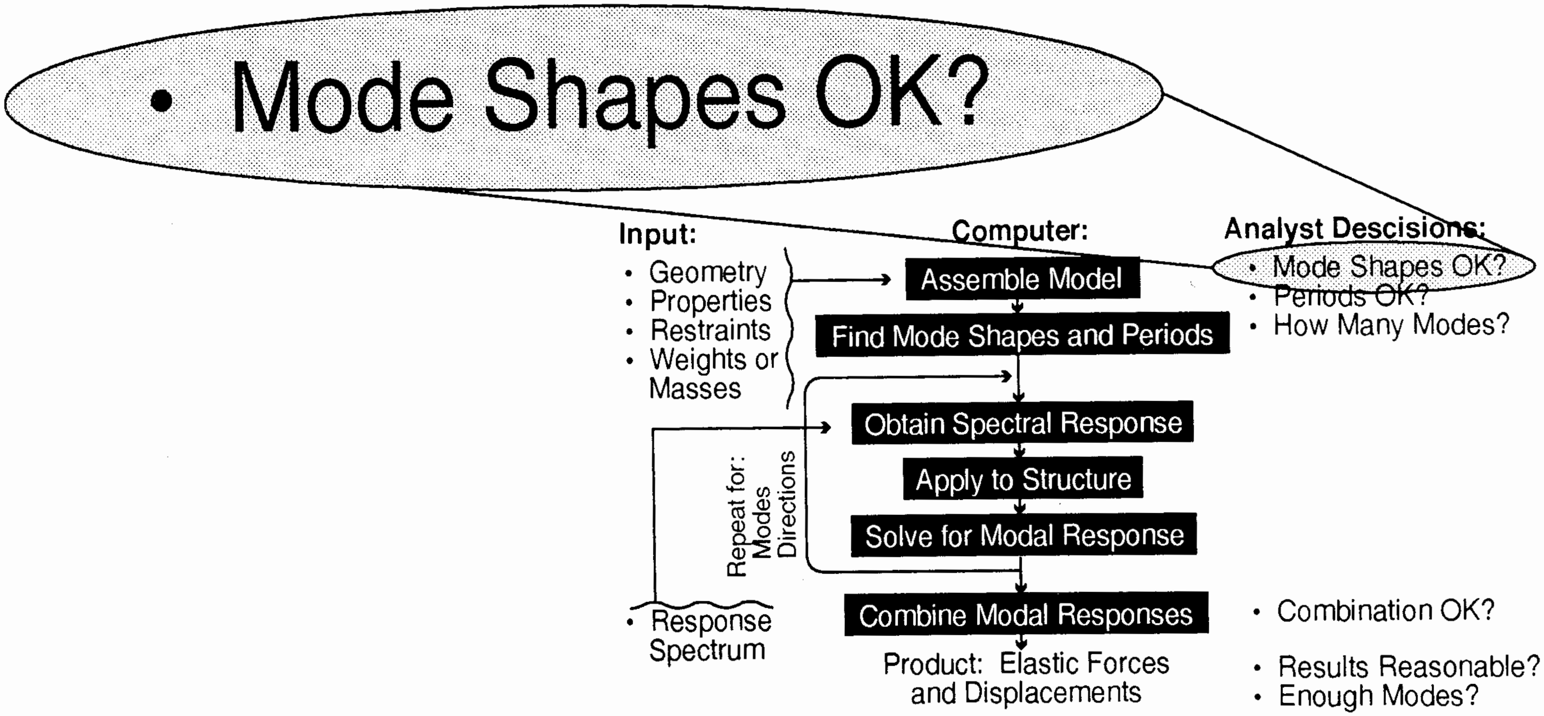
MODE NUMBER	EIGENVALUE (RAD/SEC)**2	CIRCULAR FREQ (RAD/SEC)	FREQUENCY (CYCLES/SEC)	PERIOD (SEC)
1	0.606219E+02	0.778601E+01	1.239182	0.806984
2	0.199010E+03	0.141071E+02	2.245216	0.445392
3	0.818561E+03	0.286105E+02	4.553502	0.219611
4	0.128067E+04	0.357865E+02	5.695596	0.175574
5	0.244260E+04	0.494226E+02	7.865859	0.127132
6	0.840641E+04	0.916865E+02	14.592353	0.068529
7	0.957479E+04	0.978508E+02	15.573445	0.064212
8	0.174203E+05	0.131986E+03	21.006200	0.047605
9	0.191682E+05	0.138449E+03	22.034859	0.045383
10	0.269107E+05	0.164045E+03	26.108526	0.038302
11	0.437236E+05	0.209102E+03	33.279597	0.030048
12	0.622932E+05	0.249586E+03	39.722864	0.025174
13	0.932711E+05	0.305403E+03	48.606414	0.020573
14	0.130158E+06	0.360774E+03	57.419016	0.017416
15	0.139134E+06	0.373006E+03	59.365809	0.016845

15 of 38 Total

Example Bridge – Mode Shapes

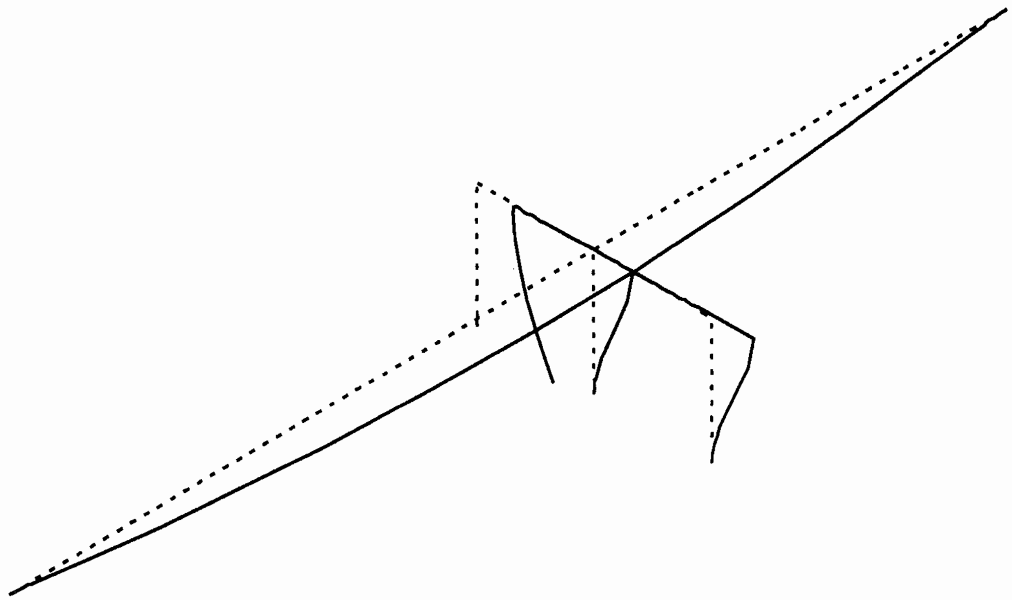


Multimode Dynamic Analysis



Checking Results with Mode Shapes

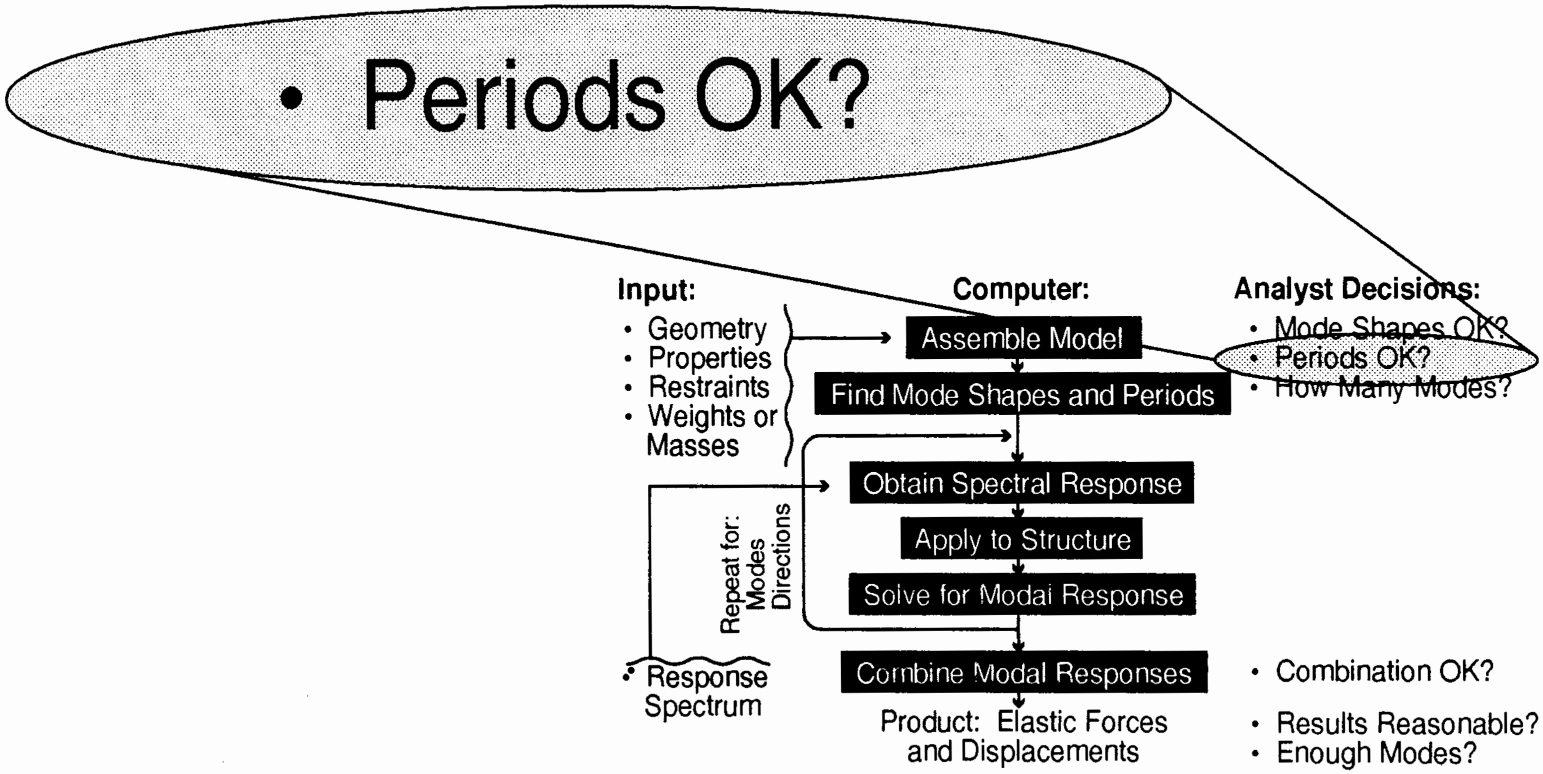
- **Inspect All Mode Shapes for Realism**



Session 6
Multimode Dynamic Analysis

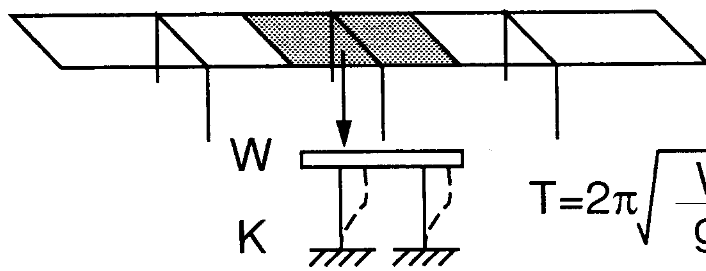
- **Continuation of Session 5**

Multimode Dynamic Analysis



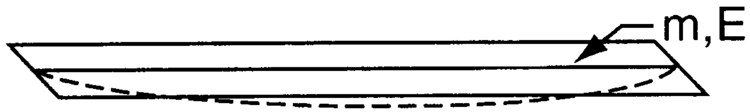
Check of Periods

- Rigid Body Structure
  (Assume Structure
  Moves as Rigid Body)



$$T = 2\pi\sqrt{\frac{W}{gK}}$$

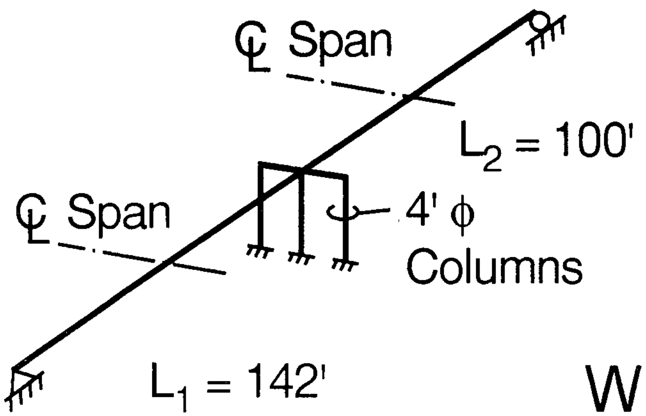
- Use Solution
  for Assumed Shape
  (See Structural Dynamics Texts)



$$T = \frac{2}{\pi}\sqrt{\frac{mL^4}{EI}}$$

Simple Supports

Example Check – Transverse / Rigid Body



$E_{col} = 518{,}420$ ksf

$I_{col} = 12.57$ ft

$H_{clr} = 27.33$ ft

W = 2438 kip
Tributary Weight (1/2 of Each Span)
Plus 1/2 Column Weight

Example Check – Transverse / Rigid Body (continued)

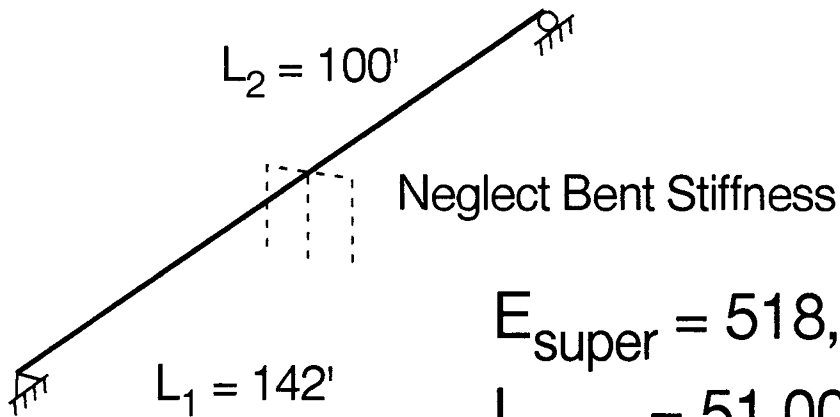
$$K = 3\left(\frac{12\,E_{col}\,I_{col}}{H^3}\right) = 3\left(\frac{12\,(518{,}400)12.57}{(27.33)^3}\right) = 11{,}500 \text{ kip/ft}$$

$$T = 2\pi\sqrt{\frac{W}{gK}} = 2\pi\sqrt{\frac{2438}{32.2(11{,}500)}} = 0.510 \text{ sec}$$

N.G.

Multimode T = 0.172 sec
Actual Behavior, More Like Simple Beam

Example Check – Transverse / Simple Beam Solution



$E_{super} = 518{,}400$ ksf

$I_{super} = 51{,}000$ ft^4

$L = 242$ ft

$W_{total} = 4876$ kip

Example Check – Transverse / Simple Beam Solution (continued)

Equivalent Distributed Mass, m

$$m = \frac{W_{total}}{gL} = \frac{4876}{32.2(242)} = 0.626 \ \frac{\text{ksec}^2}{\text{ft}^2}$$

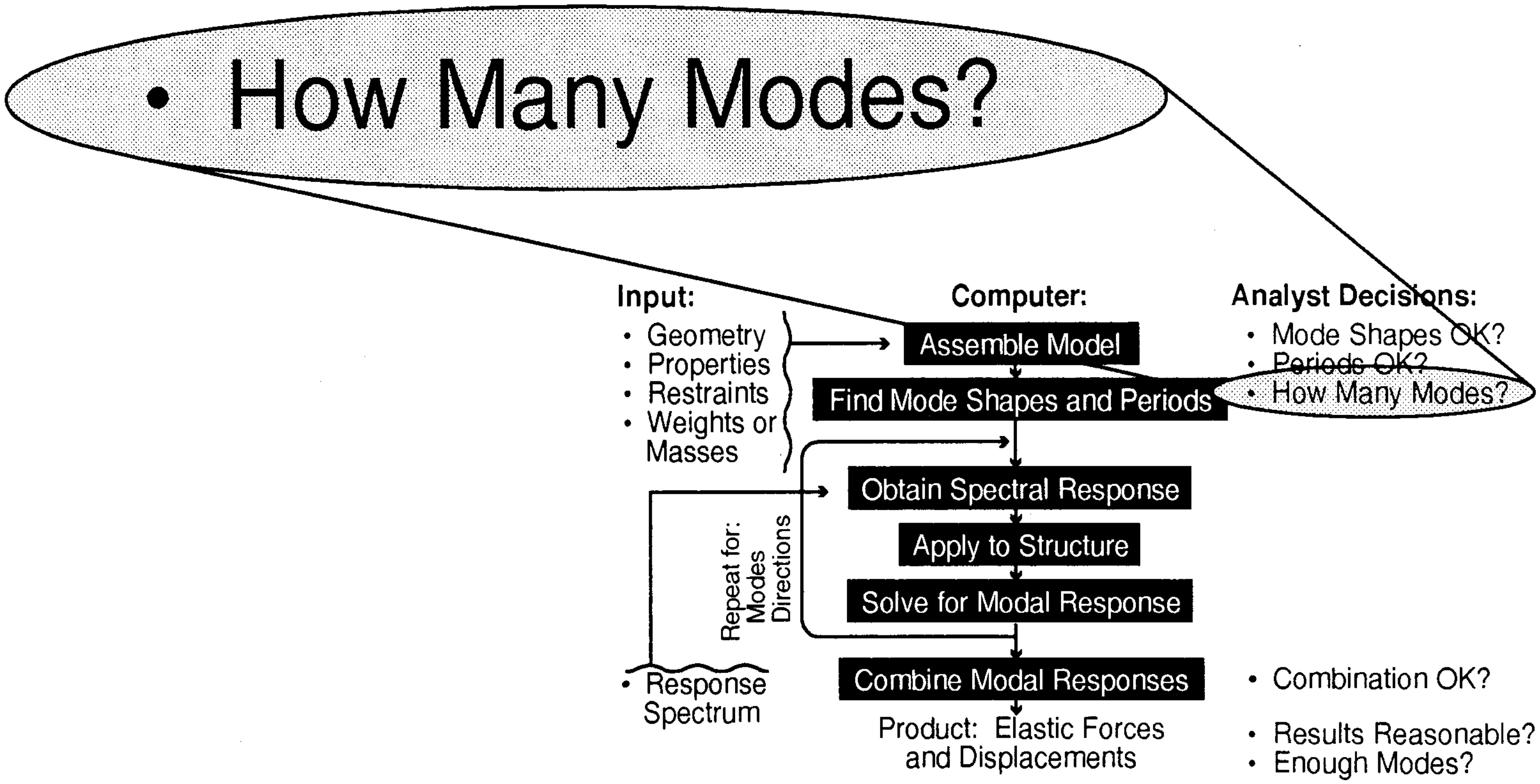
$$T = \frac{2}{\pi}\sqrt{\frac{mL^4}{EI}} = \frac{2}{\pi}\sqrt{\frac{0.626\ (242)^4}{518{,}400\ (51{,}000)}} = 0.181 \text{ sec}$$

Reference: Structural
Dynamics Texts

5% OK

Multimode T = 0.172 sec

Multimode Dynamic Analysis



Not All Modes Are Required

Response Can Be Estimated with Several Modes in Each Direction, Typically

- **AASHTO Recommends**

  3 • No. Spans ≤ 25 Modes

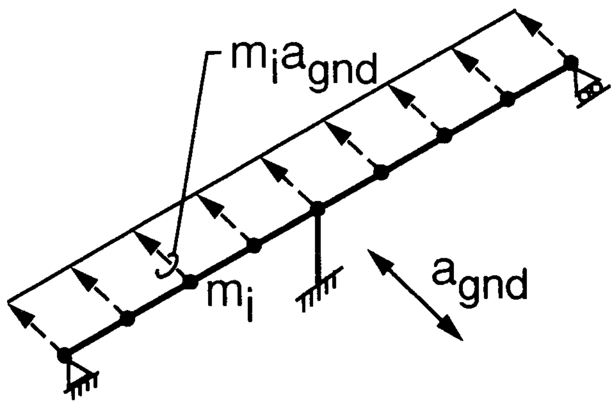
- **Other Recommendations**

  4 • No. Spans, No Upper Limit

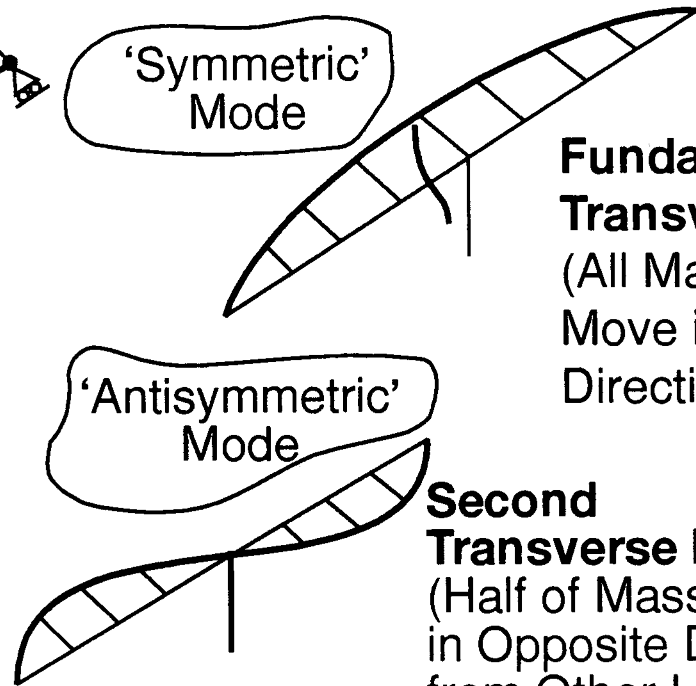
  Participating Mass, 90 – 95%

  Make Sure All Parts of Structure Move

Modal Participation



Transverse Acceleration and Loading

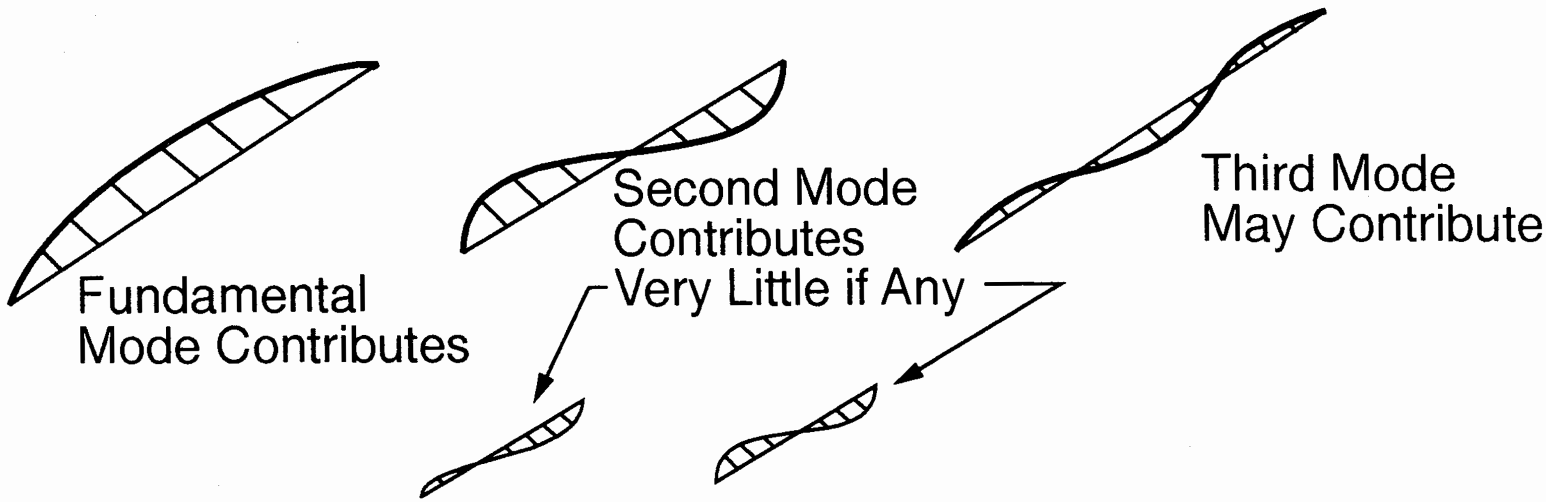


Fundamental Transverse Mode
(All Masses Move in Same Direction)

Second Transverse Mode
(Half of Masses Move in Opposite Direction from Other Half)

Modal Participation (continued)

Result: Earthquake Loading Will Tend to Excite Only Those Modes that Have a Net Translation in Earthquake Direction



Rating the Importance of Each Mode

Participating Mass, PM = Base Shear Contributed by Each Mode for a Constant Spectral Acceleration

$$PM = \frac{\beta^2}{\gamma} \frac{100}{\text{Total Weight}} \quad \text{(\% of Structure Weight)}$$

Constants — Single-Mode Method Definitions
β — Earthquake Excitation for Each Mode
γ — Effective Weight for Each Mode

How Many Modes Are Required?

- In Our Example, There Are **38 Modes,** Total
- The First 15 Modes Have Been Determined

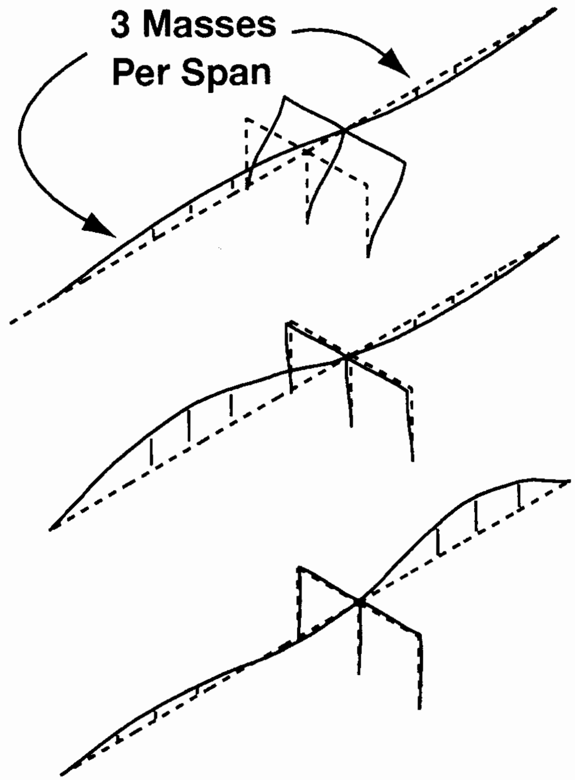
P A R T I C I P A T I N G   M A S S - (percent)

MODE	X-DIR	Y-DIR	Z-DIR	X-SUM	Y-SUM	Z-SUM	
1	93.148	0.646	0.000	93.148	0.646	0.000	
2	4.807	26.201	0.000	97.955	26.847	0.000	
3	0.322	45.473	0.000	98.277	72.321	0.000	
4	0.000	0.000	88.890	98.277	72.321	88.890	
5	0.032	4.434	0.000	98.309	76.754	88.890	
6	0.013	0.000	0.000	98.322	76.755	88.890	← AASHTO (3x Spans)
7	0.000	7.999	0.000	98.322	84.754	88.890	
8	0.000	0.000	0.000	98.322	84.754	88.890	← 4x Spans
9	0.000	0.000	0.000	98.322	84.754	88.890	
10	0.000	12.396	0.000	98.322	97.151	88.890	
11	0.001	1.149	0.000	98.323	98.300	88.890	
12	0.000	0.000	0.000	98.323	98.300	88.890	
13	0.000	0.000	7.103	98.323	98.300	95.993	← 90-95%
14	0.000	0.000	0.000	98.323	98.300	95.993	
15	0.000	0.000	0.000	98.323	98.300	95.993	

Example Bridge / Participating Mass

Participating Mass (%)

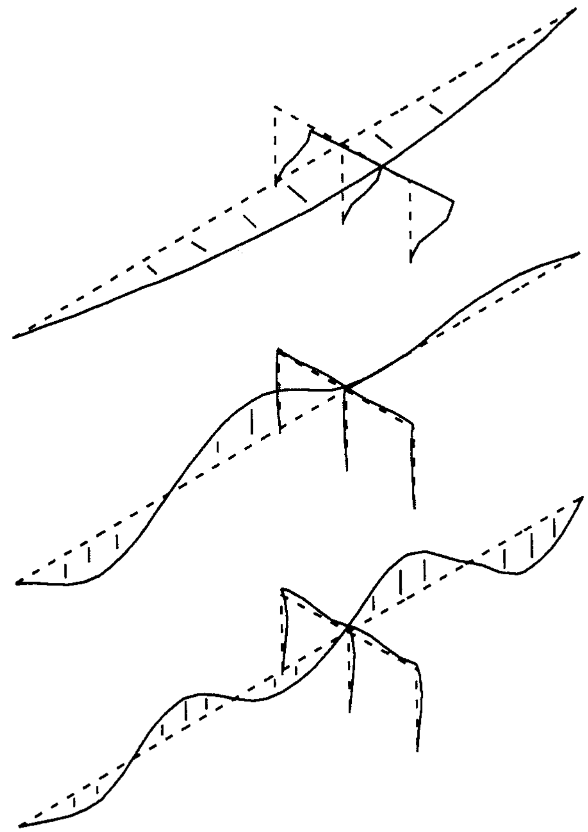
Mode	Longitudinal	Vertical	Transverse
1	93.2	0.6	0.0
	(First Longitudinal Mode)		
2	4.8	26.2	0.0
	(First Vertical Mode)		
3	0.3	45.5	0.0
	(Second Vertical Mode)		



Example Bridge / Participating Mass (continued)

Participating Mass (%)

Mode	Longitudinal	Vertical	Transverse
4	0.0	0.0	88.9
	(First Transverse Mode)		
5	0.03	4.4	0.0
6	0.01	0.0	0.0

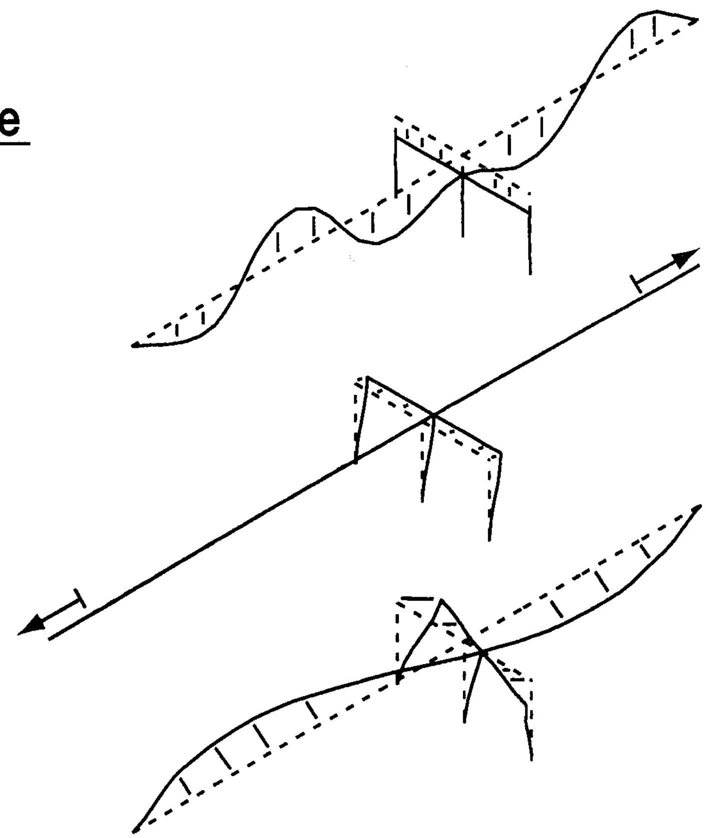


Example Bridge / Participating Mass (continued)

Participating Mass (%)

Mode	Longitudinal	Vertical	Transverse
7	0.0	8.0	0.0
8	0.0	0.0	0.0
9	0.0	0.0	0.0

(Second Transverse Mode)

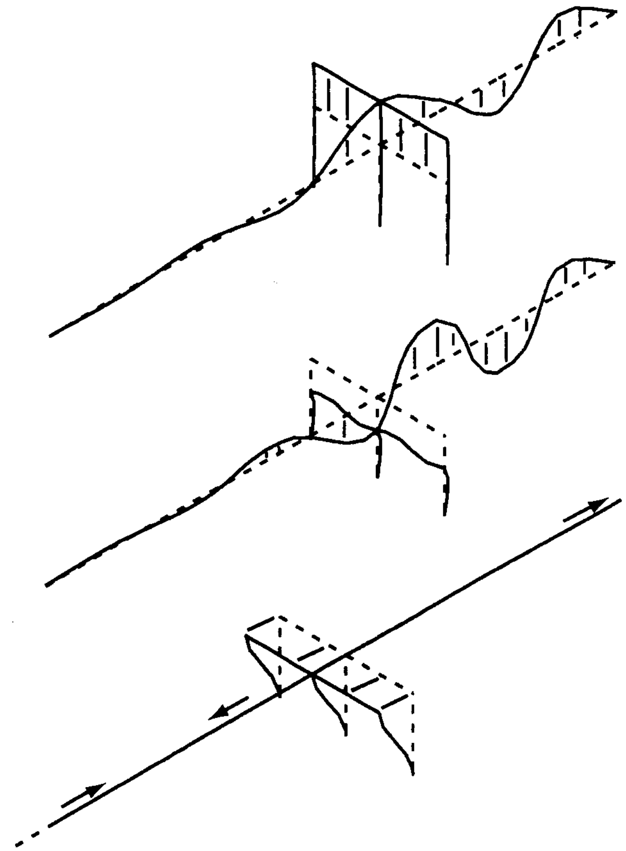


Example Bridge / Participating Mass (continued)

Participating Mass (%)

Mode	Longitudinal	Vertical	Transverse
10	0.0	12.4	0.0
11	0.0	1.1	0.0
12	0.0	0.0	0.0

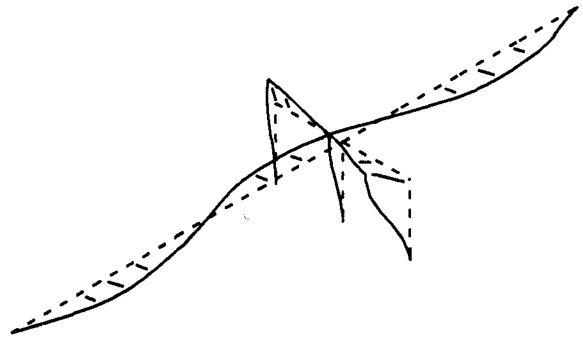
(Second Longitudinal Mode)



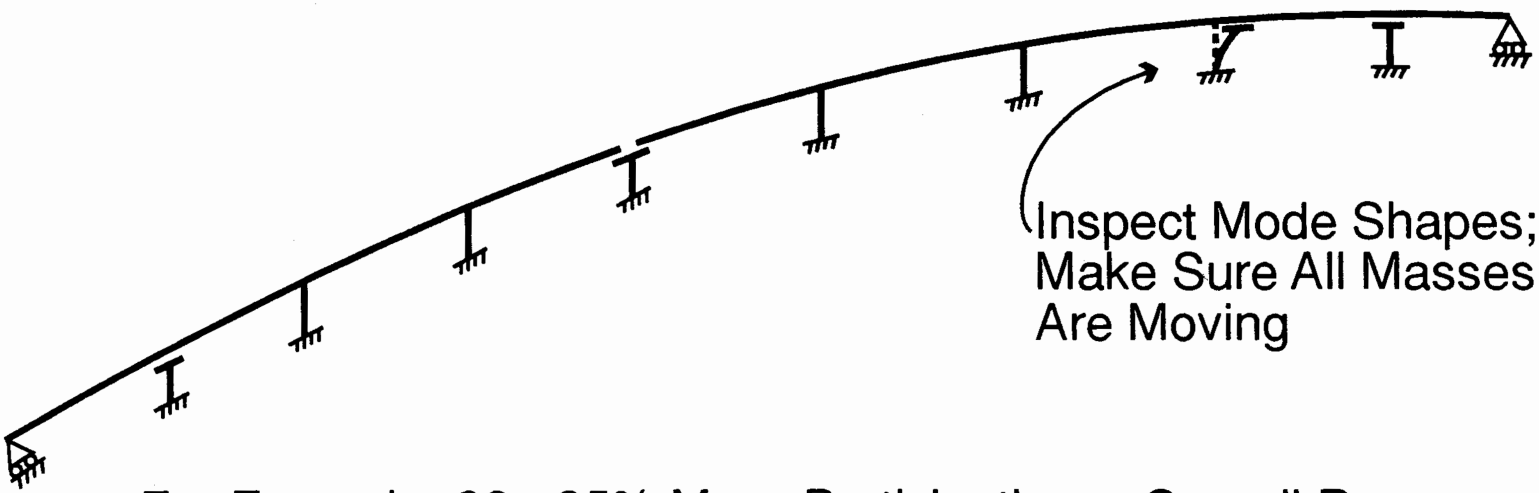
Example Bridge / Participating Mass (continued)

Participating Mass (%)

Mode	Longitudinal	Vertical	Transverse
13	0.0	0.0	7.1
	(Third Transverse Mode)		
Totals	98.3%	98.3%	96.0%



Global vs. Local Response Considerations

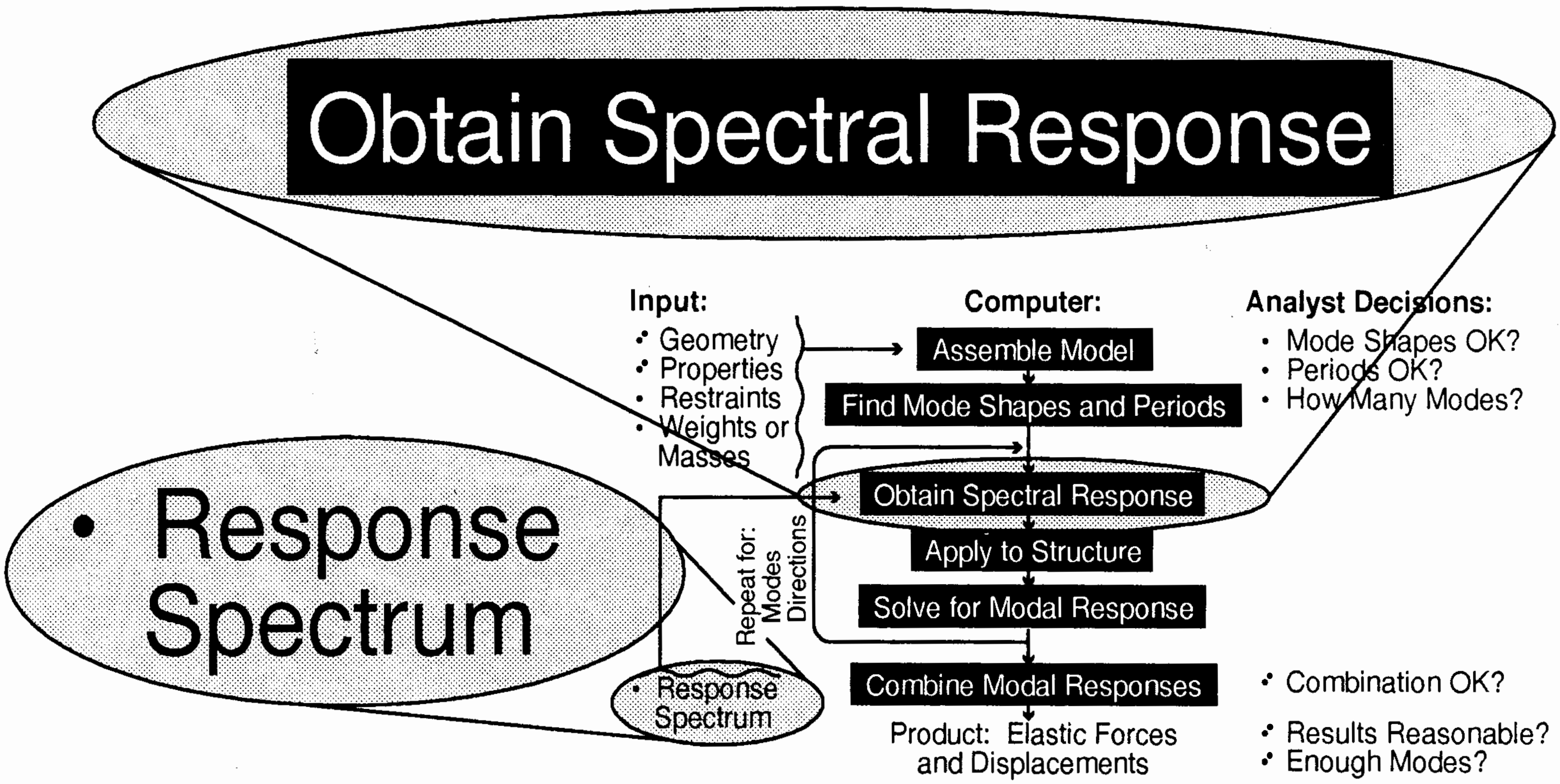


Inspect Mode Shapes;
Make Sure All Masses
Are Moving

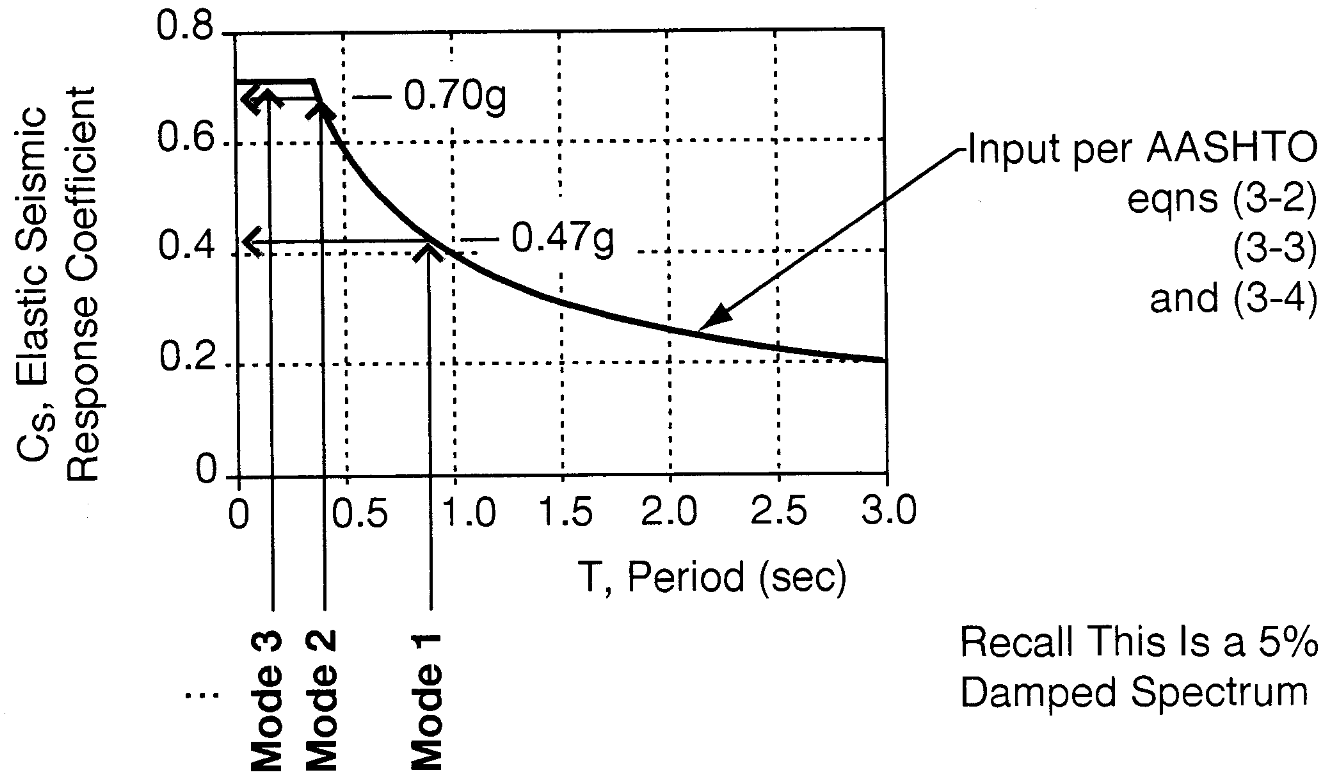
For Example: 90 - 95% Mass Participation → Overall Response Adequate

However: Additional Modes May Have Large Impact on Local Response ... Say Forces at a Given Pier

Multimode Dynamic Analysis

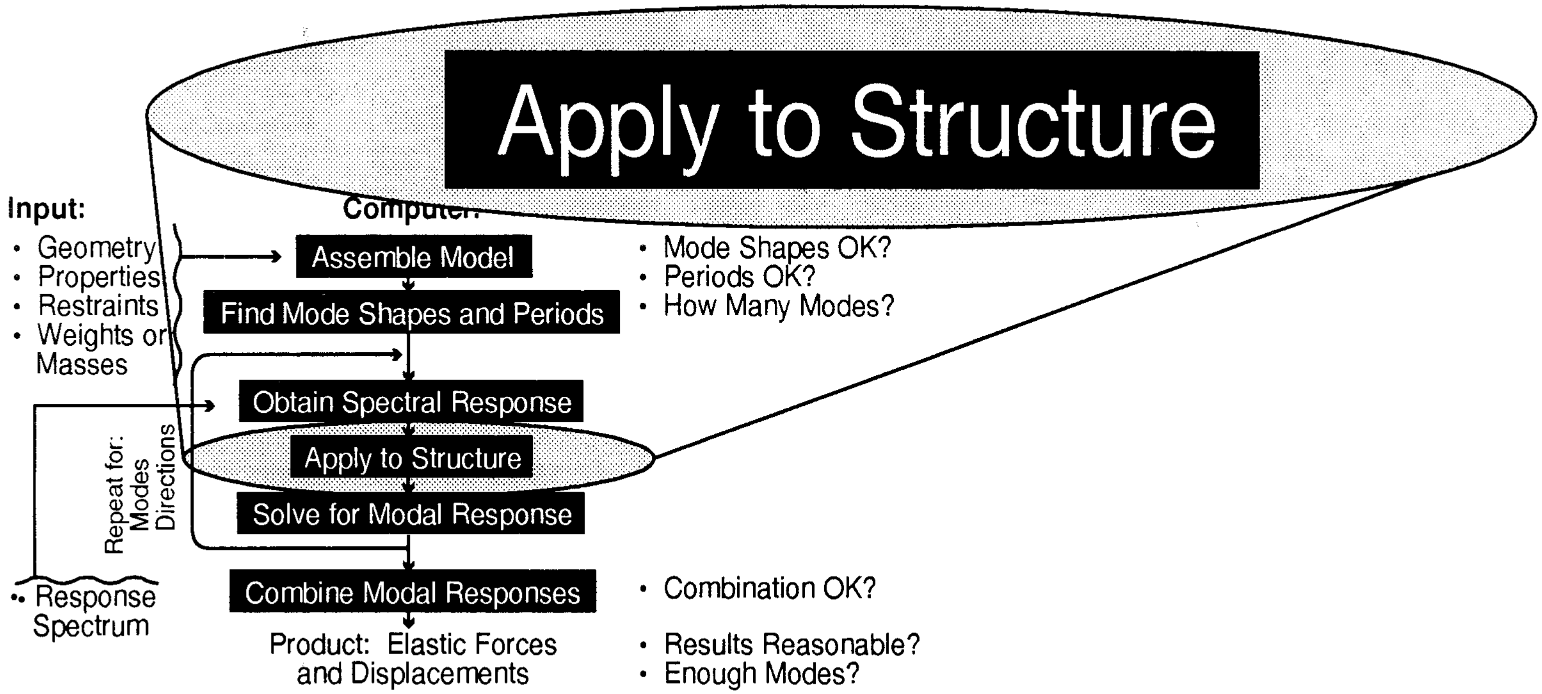


AASHTO Response Spectrum / Example Bridge



Recall This Is a 5% Damped Spectrum

Multimode Dynamic Analysis



Weighting Factors for Each Mode

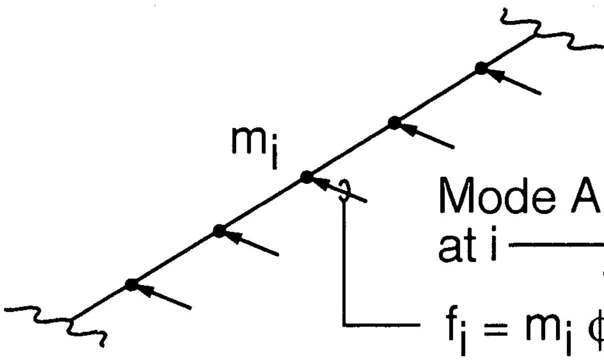
Modal Participation Factor, MPF

$$MPF = \frac{\beta}{\gamma}$$

Scales Each Mode's Contribution
(Analogy: How Many Parts of Color for Paint?)

Same as Single-Mode Factors

Apply Forces to Structure



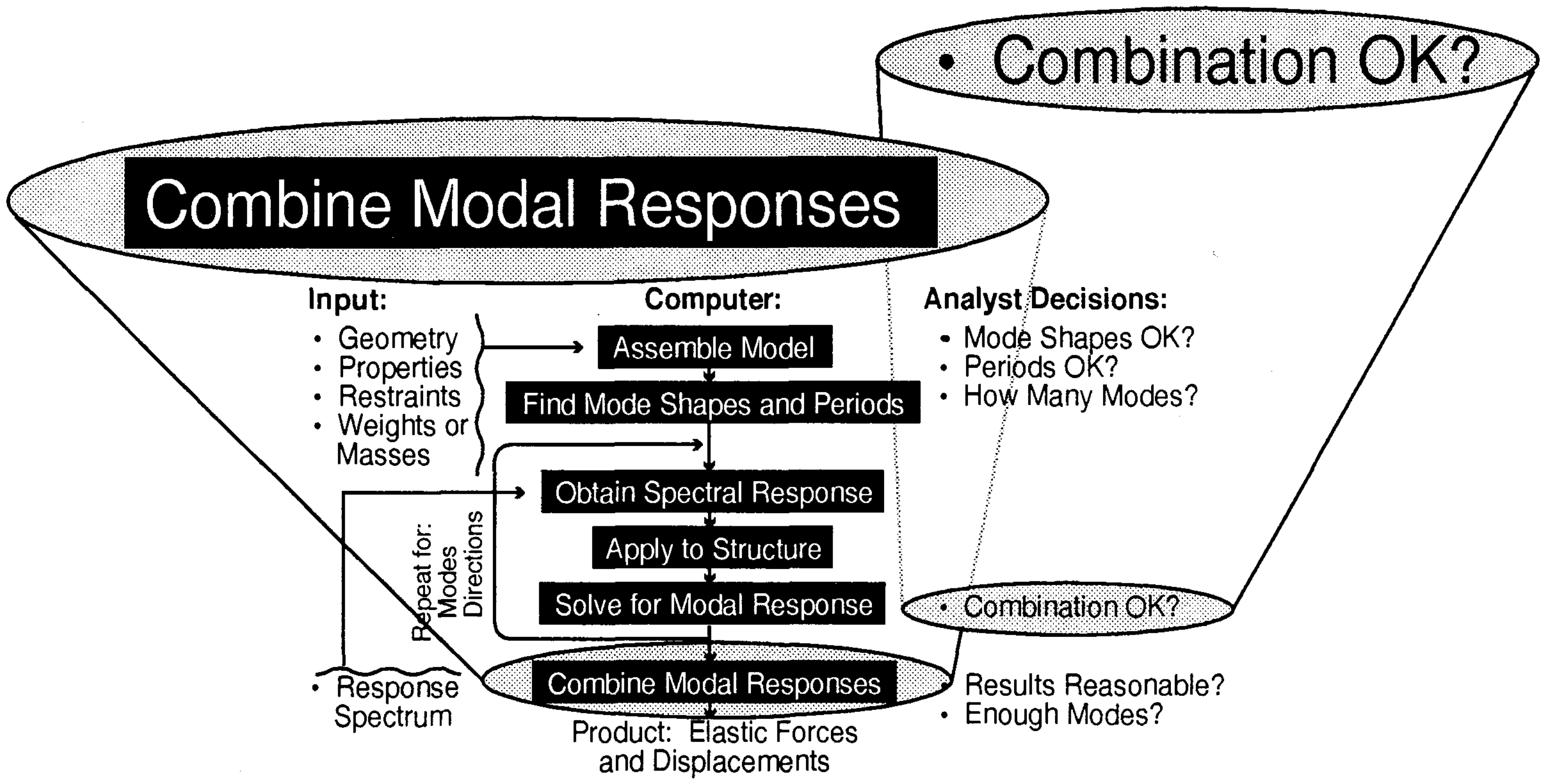
Force Applied at i

Mode Amplitude at i

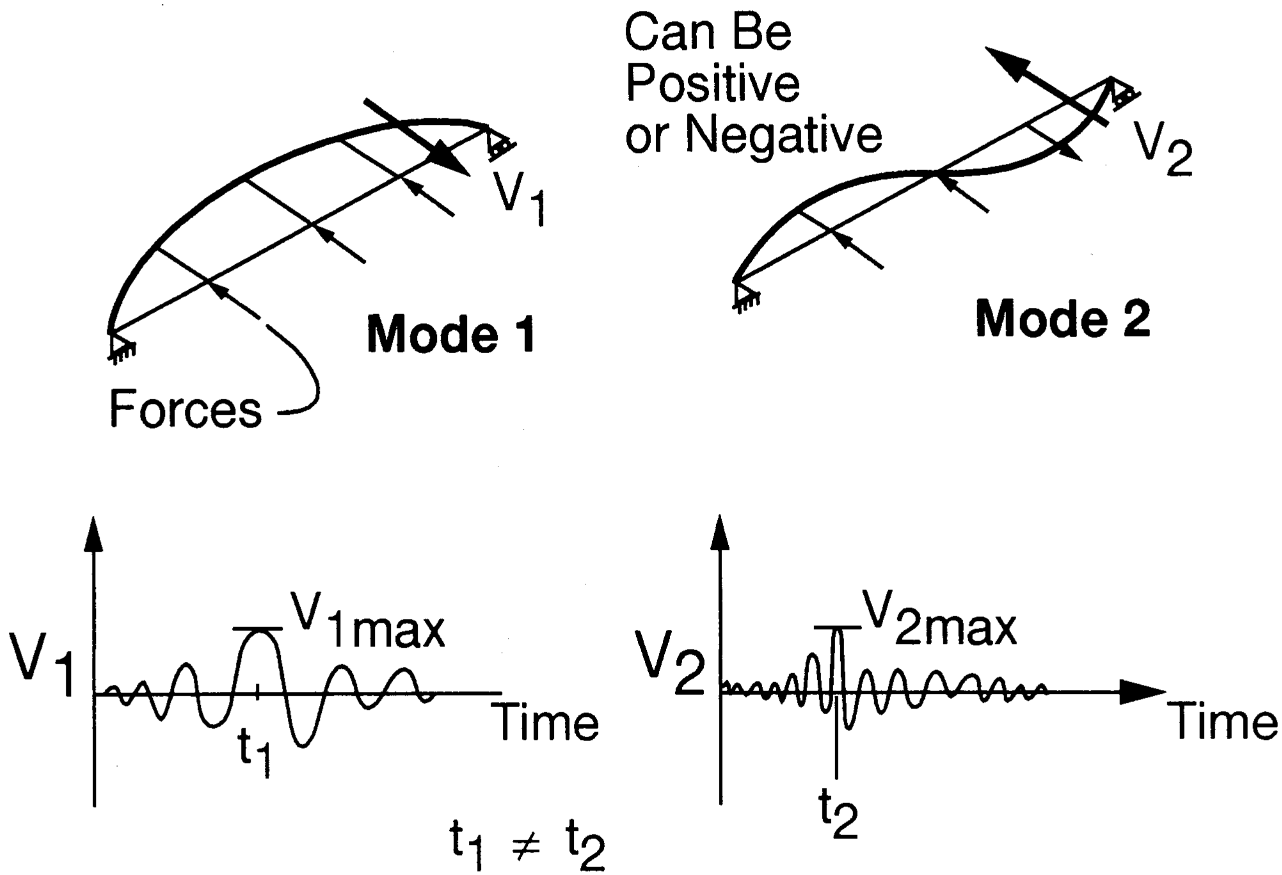
SDOF Acceleration (Fraction of g)

$$f_i = m_i \, \phi_i \, MPF \; C_{sm} g$$

Multimode Dynamic Analysis



Combining Modal Forces



Combining Modal Responses

- Since Maxima Do Not Occur at the Same Time, Adding Results May Be too Conservative

  'SAV' — Sum of Absolute Values, too Big

- Could Use
  'SRSS' — Square Root of Sum of the Squares

$$V = \sqrt{V_1^2 + V_2^2 + \ldots}$$

OK, if Periods Are Well Separated

Combining Modal Responses (continued)

- Recommend

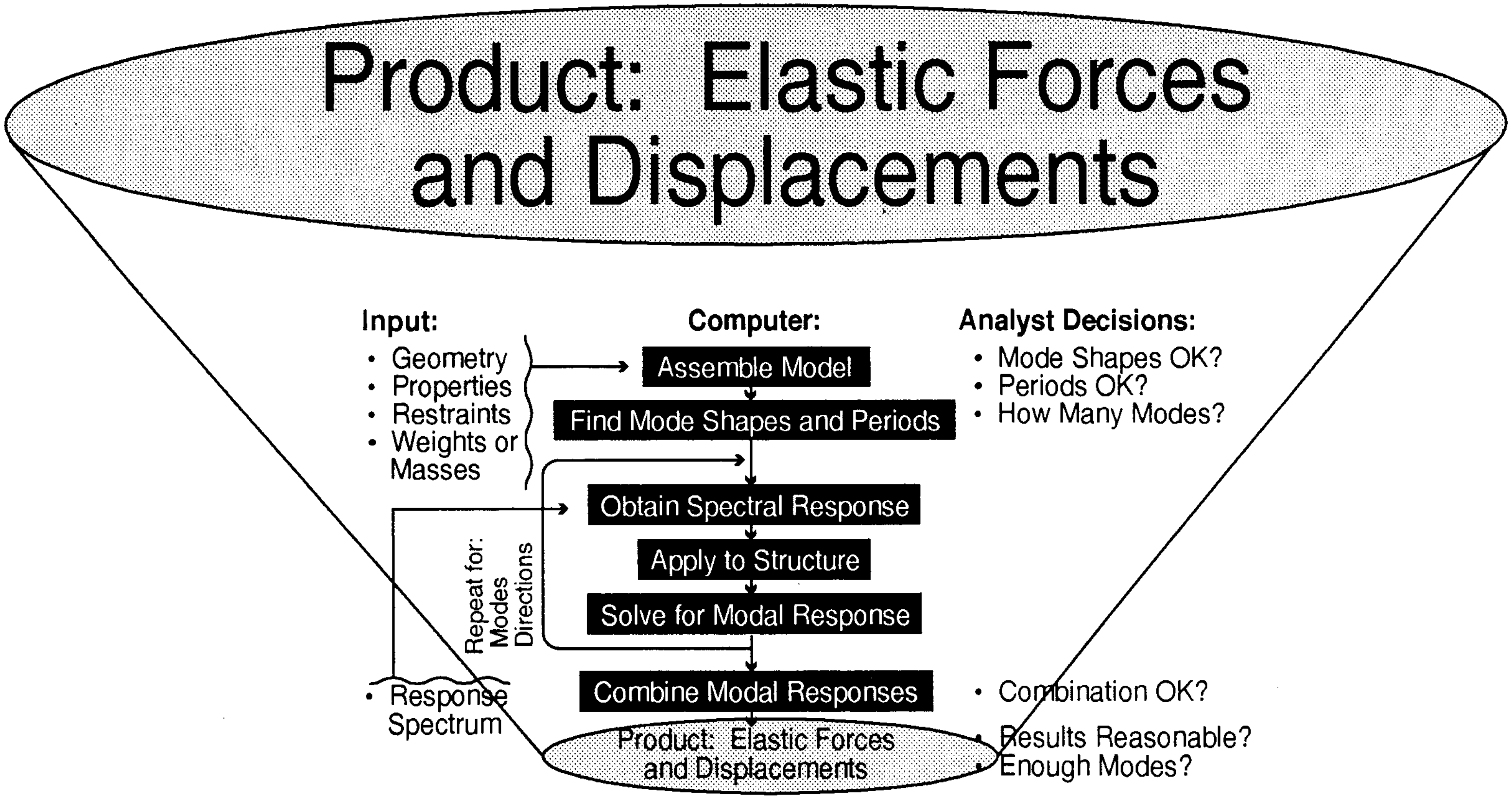
  'CQC' — Complete Quadratic Combination

  Handles Interaction of Modal Response when Periods Are Close

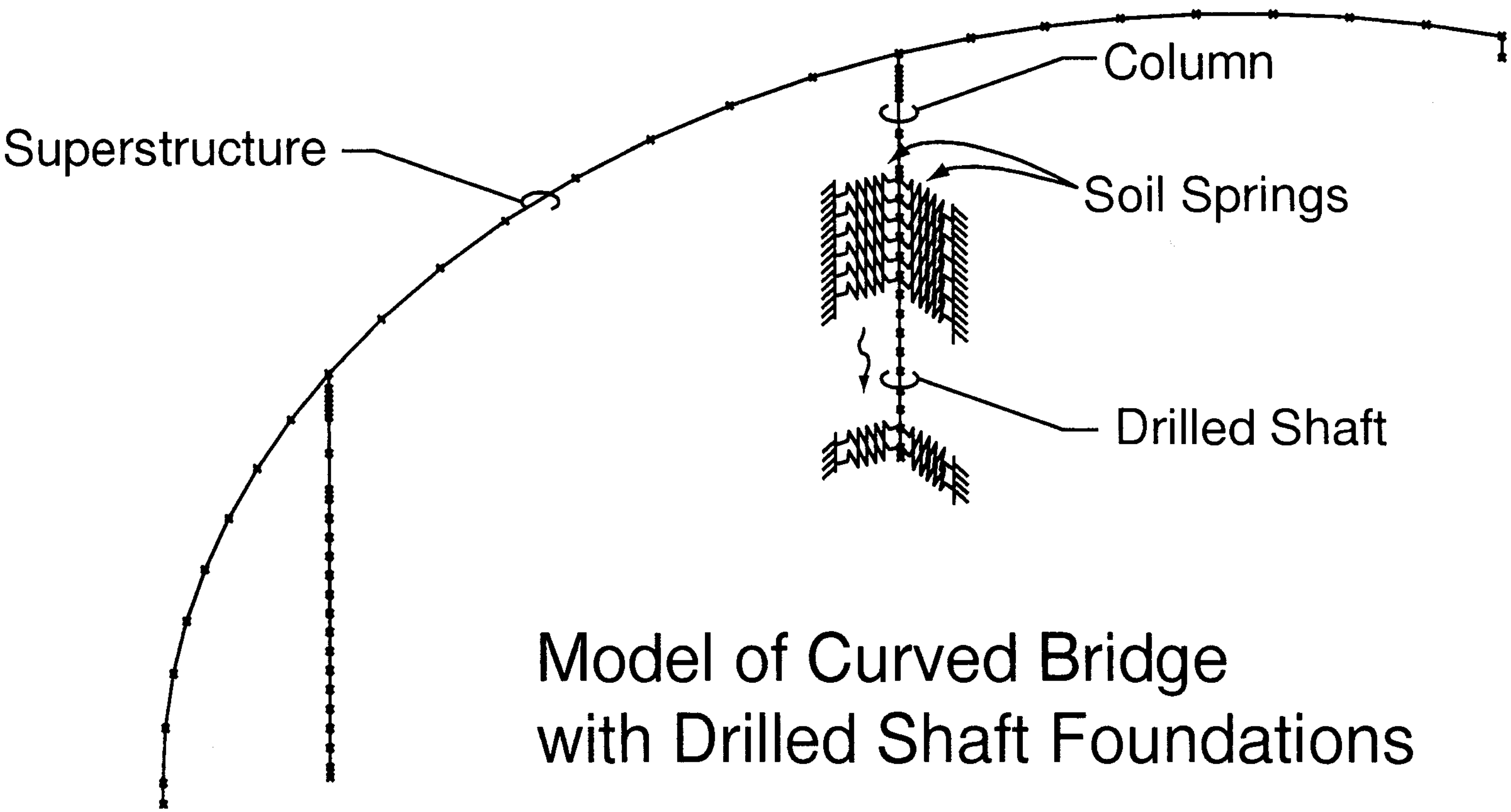
  CQC Turns into SRSS for Well-Spaced Modes

  'Square Root of Sum of the Squares'

Multimode Dynamic Analysis

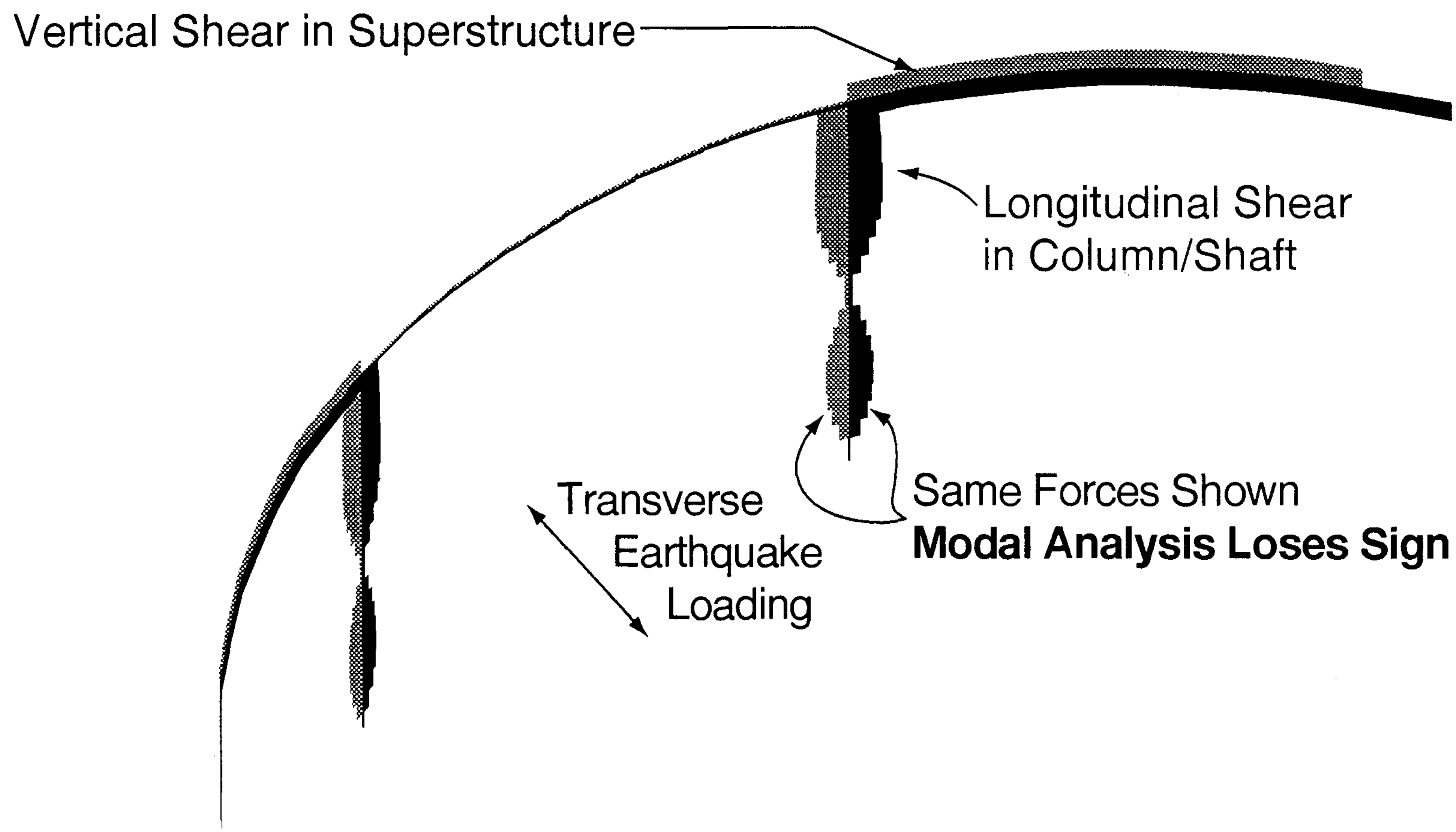


More Complex Example / Curved Bridge

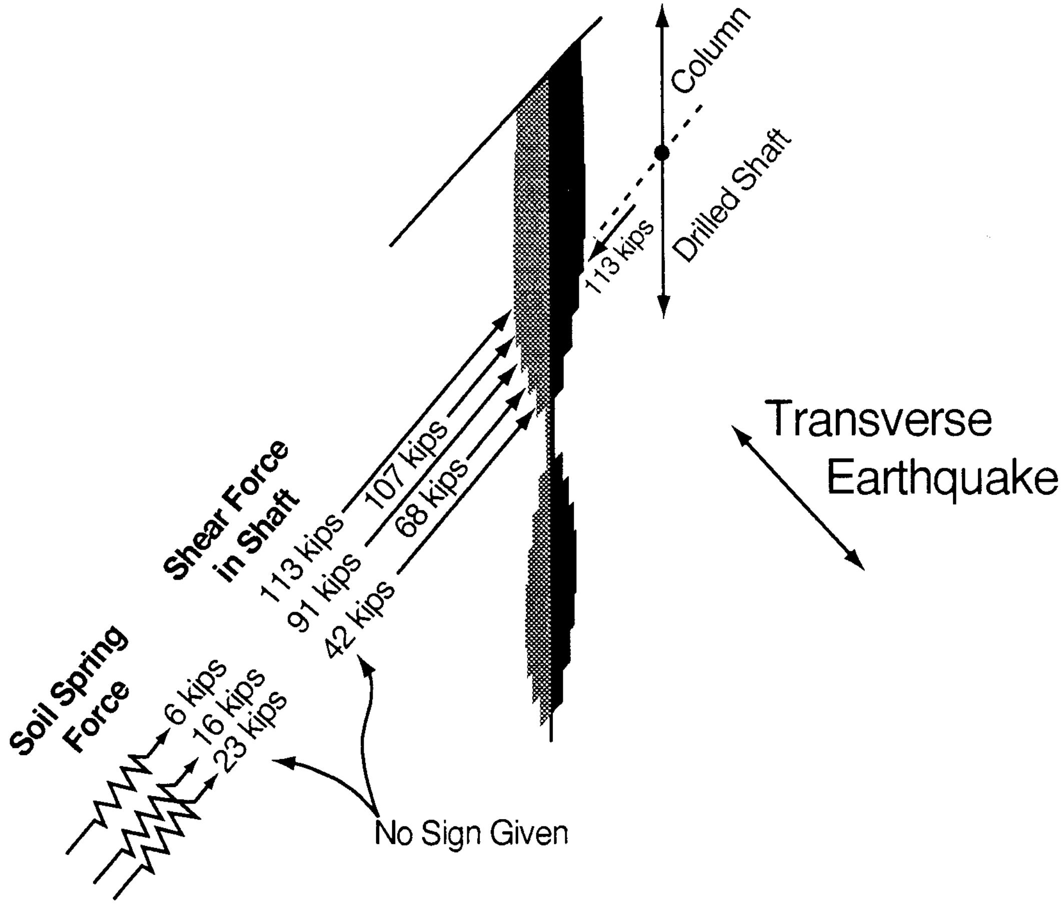


Model of Curved Bridge
with Drilled Shaft Foundations

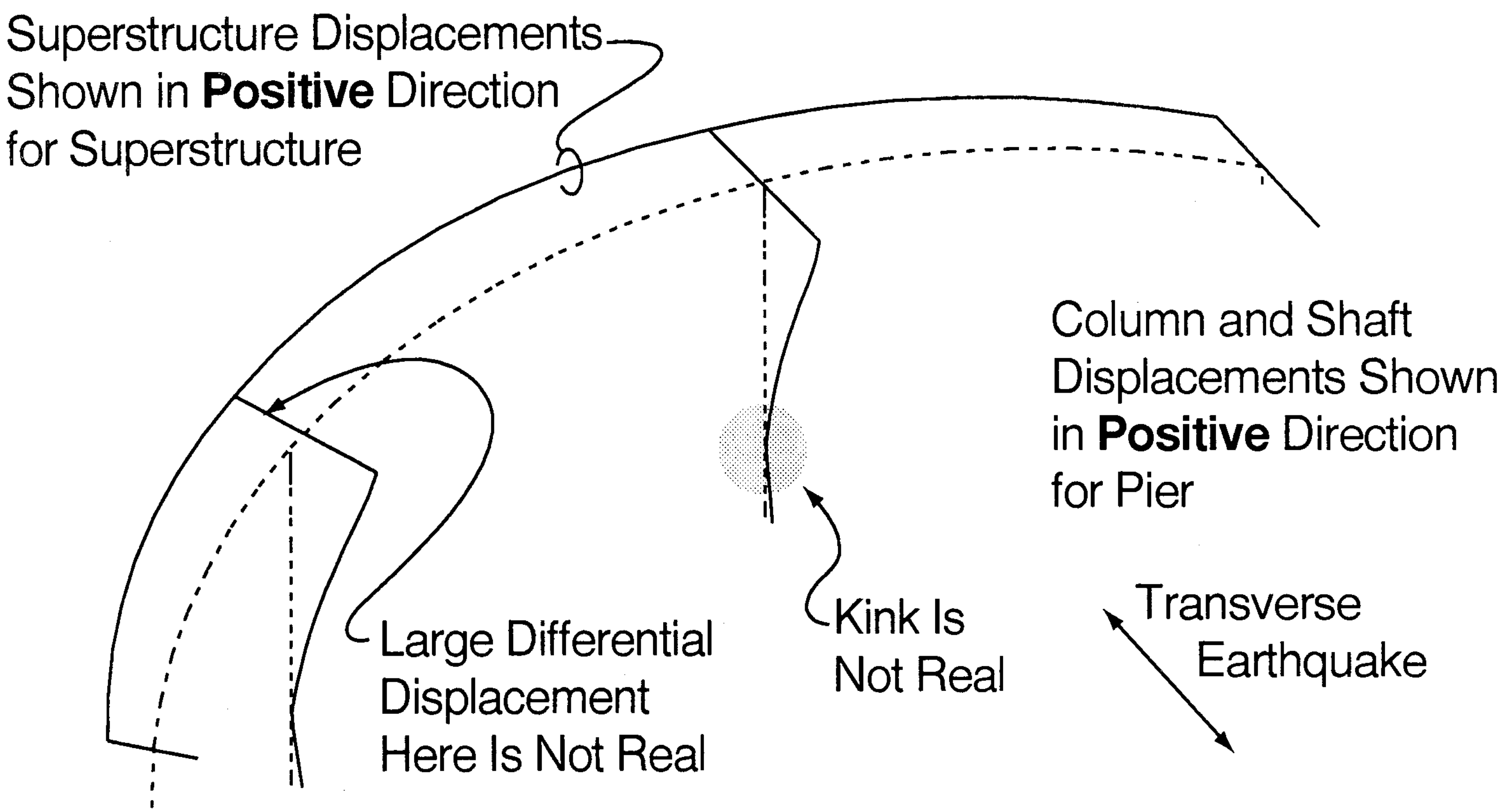
Multimode Shear Forces



Zoom in on Column / Shaft



Multimode Displacements



Interpretation of Results

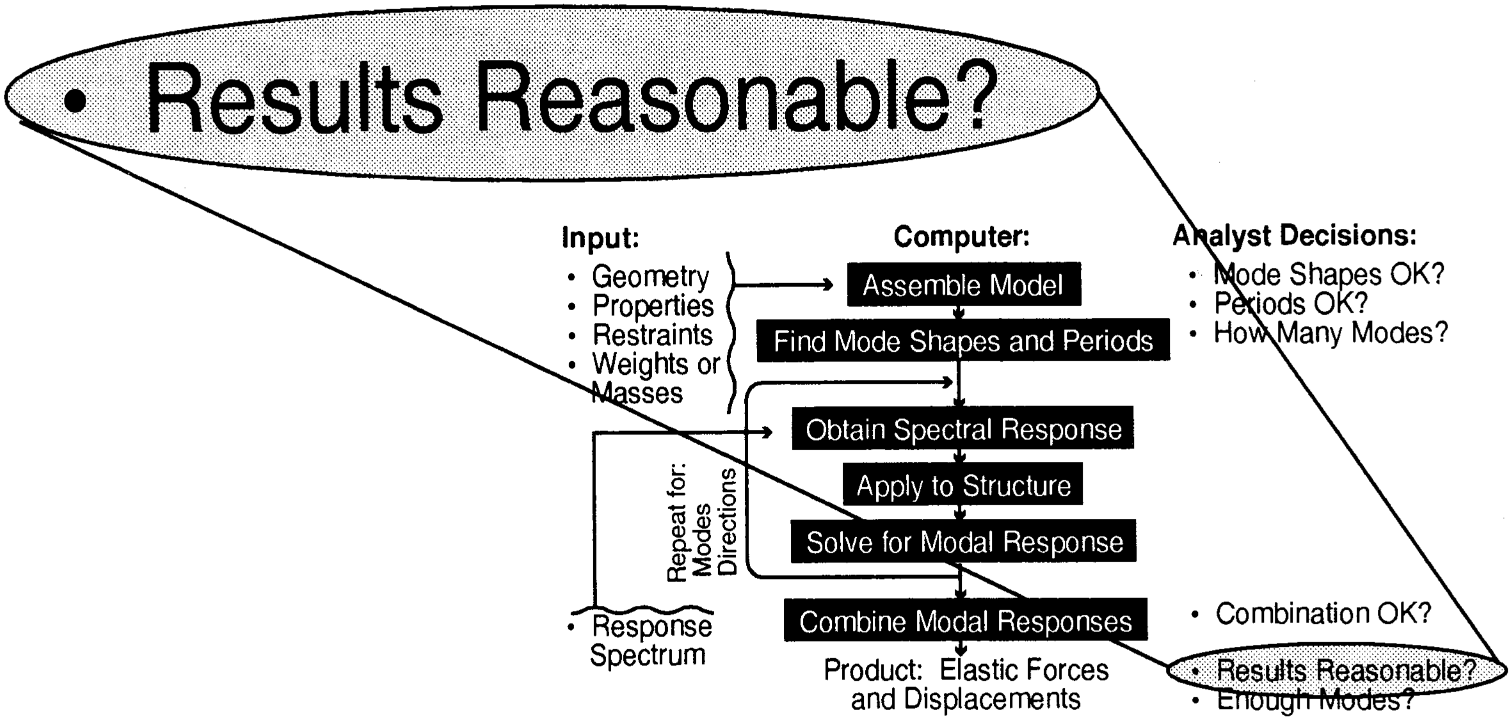
- Forces, Reactions, and Displacements Are Reported as Positive

$$\left(\sqrt{F^2} \right)$$

- Due to Loss of Sign, Equilibrium Checks Are Difficult or Impossible

- Statics Checks Are Possible on a Mode-by-Mode Basis (i.e., Each Mode Separately)

Multimode Dynamic Analysis



Example – Check Total Transverse Shear

Could Use:

$V = C_s W \qquad W = 4876 \text{ kip}$

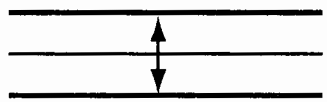
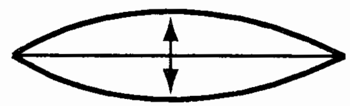
Controls

$$C_s = \frac{1.2\,AS}{T^{2/3}} = \frac{1.2\,(0.28)1.2}{(0.181)^{2/3}} = 1.26 \leq 2.5\,A = 2.5(0.28) = 0.70$$

$V = 0.70(4876) = 3413$ kip Total Shear

20% High

$V_{multimode} = 2735$ kip

Reason $\left\{ \text{Check} \right.$  Actual 

Example – Check Total Shear (continued)

Or Use Simple Beam Solution:

$$V_s(x) = \sin\frac{\pi x}{L}$$

$$\beta = \int_0^L w \sin\frac{\pi x}{L}\,dx = \frac{w2L}{\pi}$$

$$\gamma = \int_0^L w \sin^2\frac{\pi x}{L}\,dx = \frac{wL}{2}$$

$$V = PM \bullet C_s$$

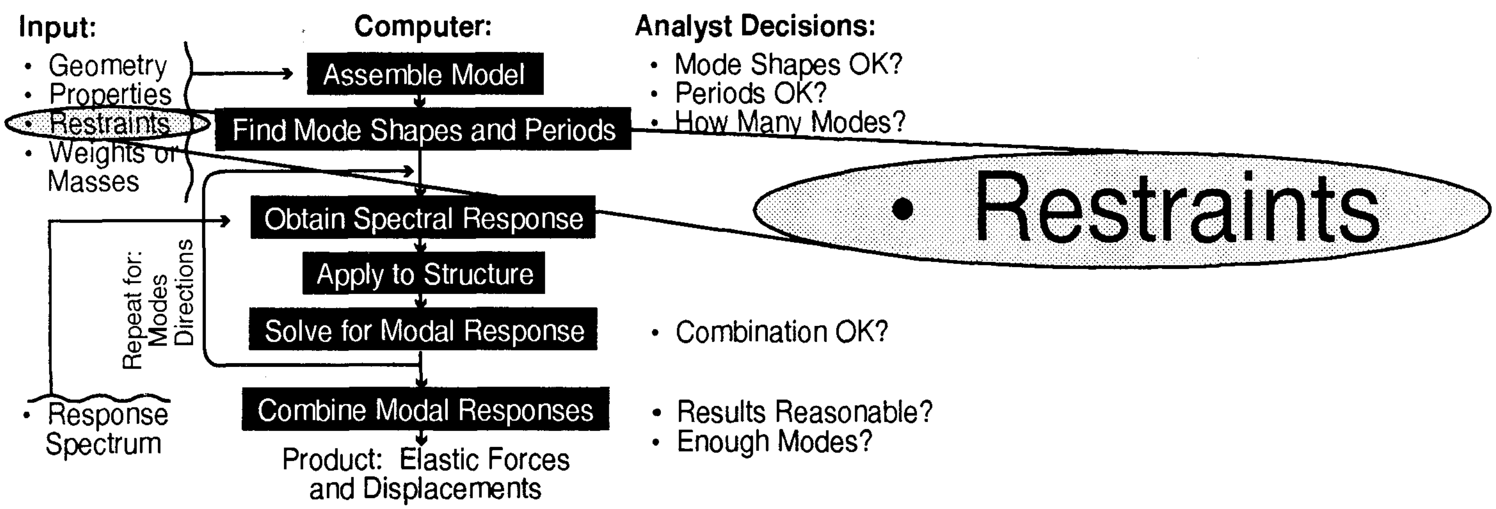
$$V = \frac{\beta^2}{\gamma} \bullet C_s = \frac{w^2 4L^2}{\pi^2}\,\frac{2}{wL} \bullet C_s = wL\frac{8}{\pi^2} \bullet C_s = W(0.811)C_s$$

$$V = 4876\ (0.811)\ 0.70 = 2767 \text{ kip}$$

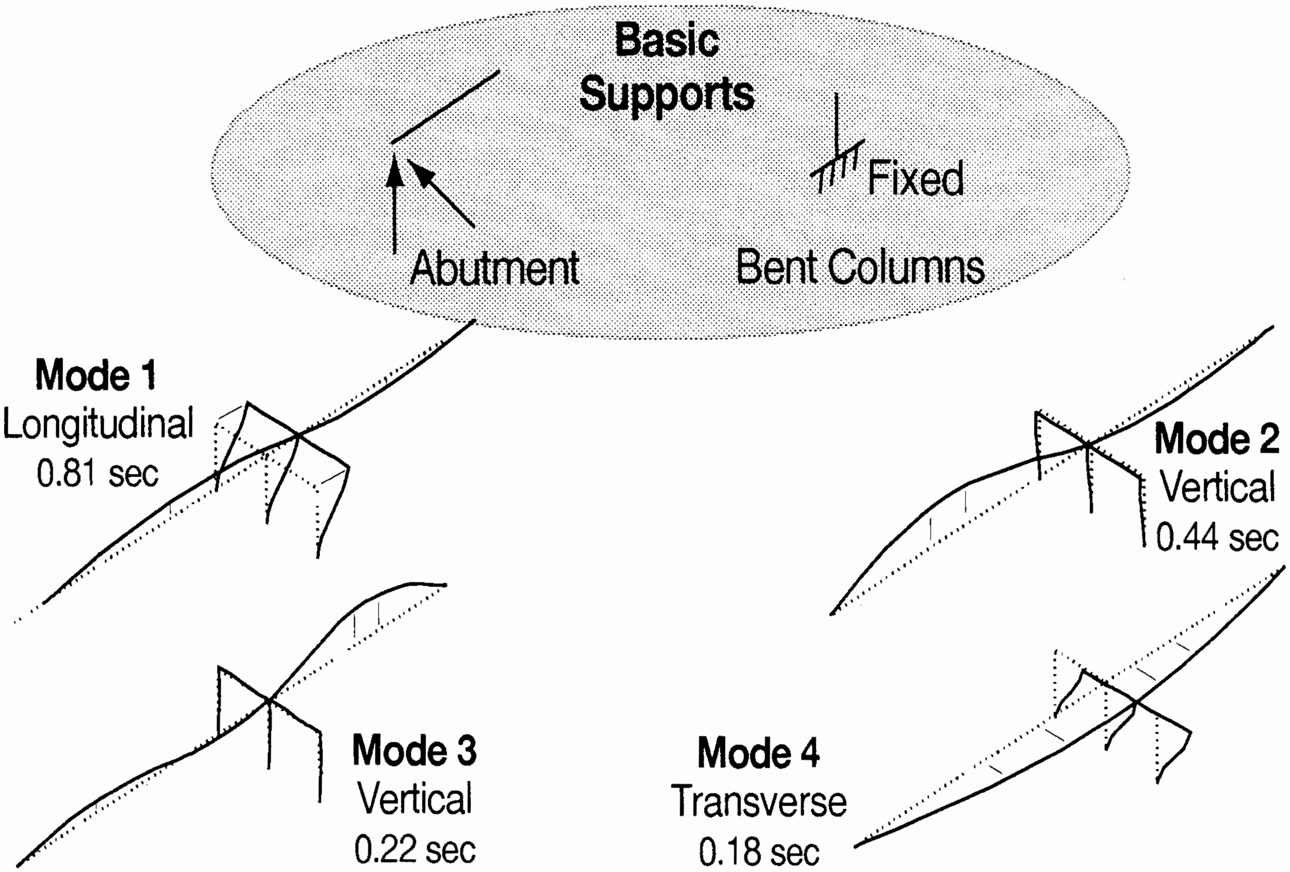
$$V_{multimode} = 2735 \text{ kip}$$

1% Difference ✔

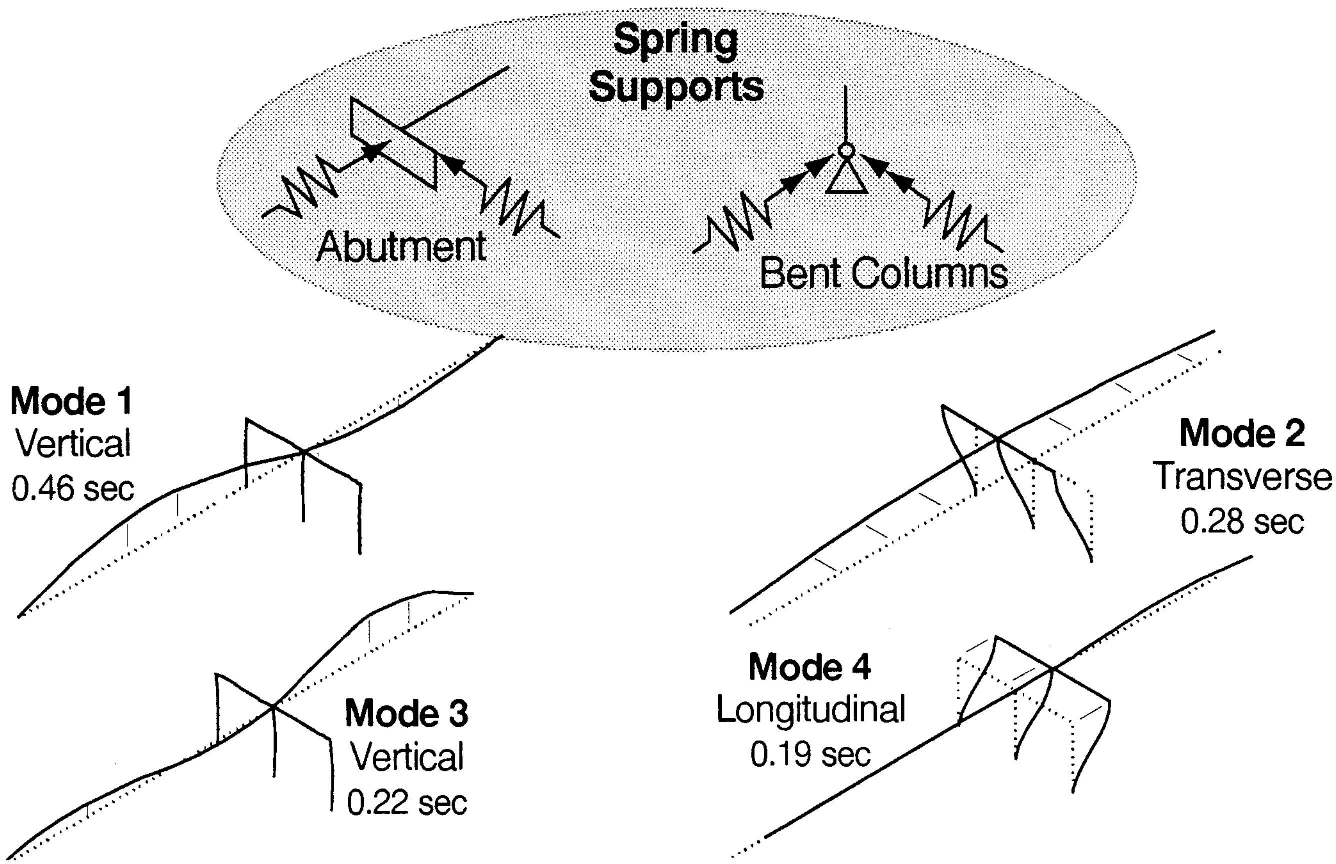
Multimode Dynamic Analysis



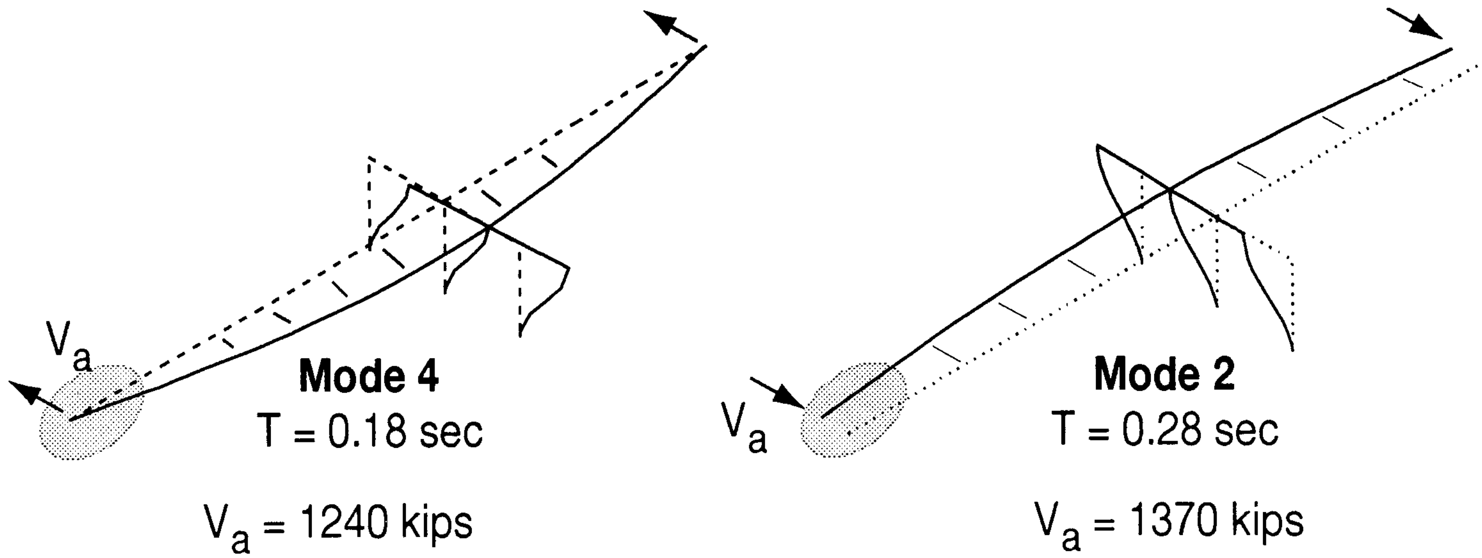
Effects of Support Conditions on Mode Shapes



Effects of Support Conditions on Mode Shapes



Effects of Support Conditions on Forces

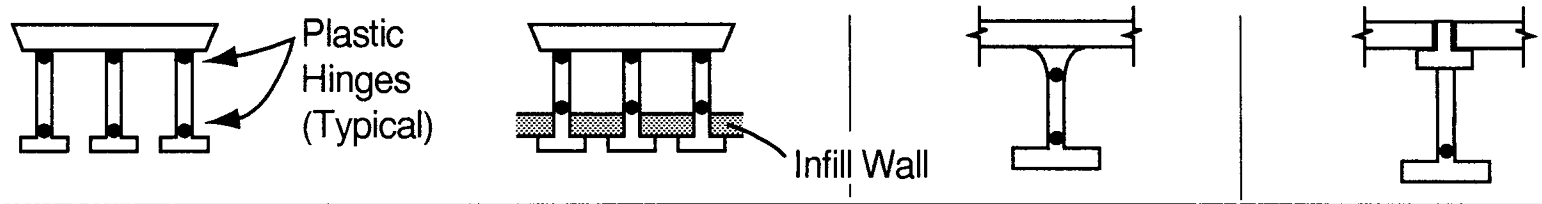


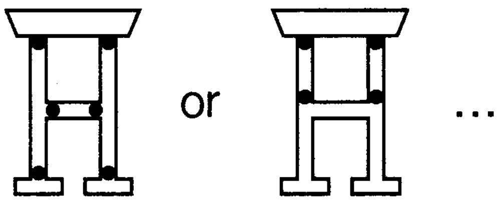
- Shear May Increase with Springs Even though Period Is Longer
- **Reason:** Superstructure Is Moving More, which Increases Inertial Forces

Session 7
Column and Pier Design

- Intended Seismic Behavior
- SPC B vs. SPC C and D Requirements
- Wall Pier Design
- General Detailing Issues

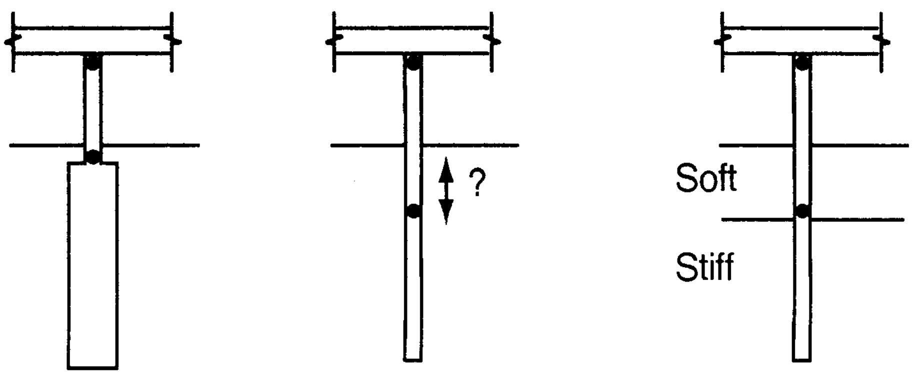
Plastic Hinging Locations / Mechanism



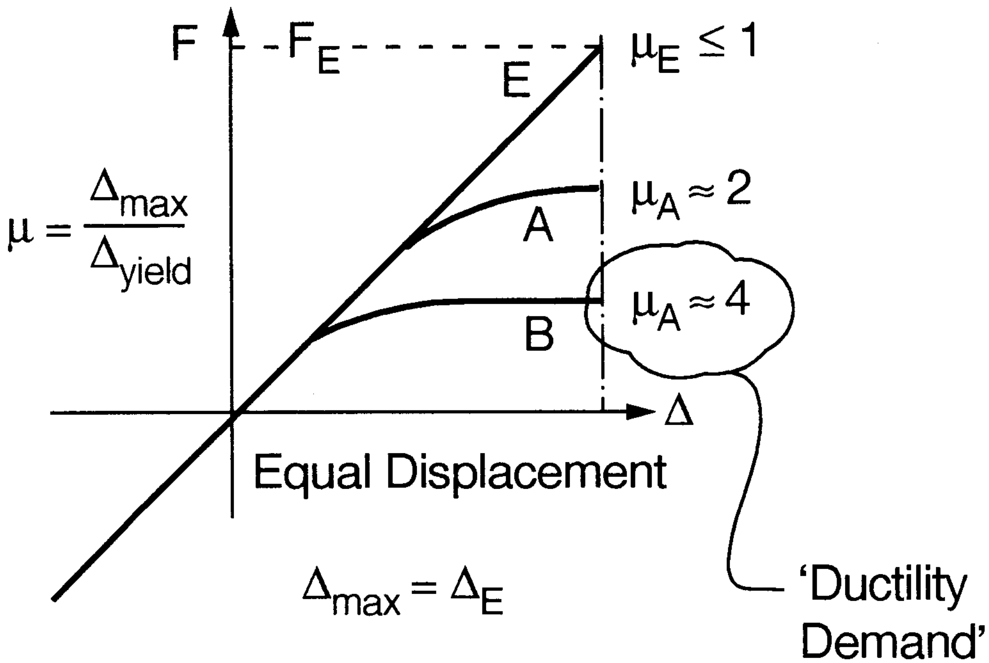


Trial and Error

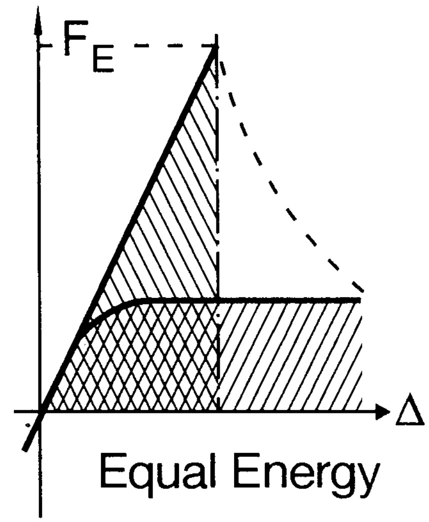
'Upper Bound Theorem' — The Lateral Force for an Assumed Mechanism Is Greater than or Equal to the True Force



Consequences of Allowing Yielding in Structures



(Medium Stiff to Flexible Structures, $0.5 < T < 3$ sec)



$\Delta_{max} = \Delta_E$

(Stiff Structures, $0.1 < T < 0.5$ sec)

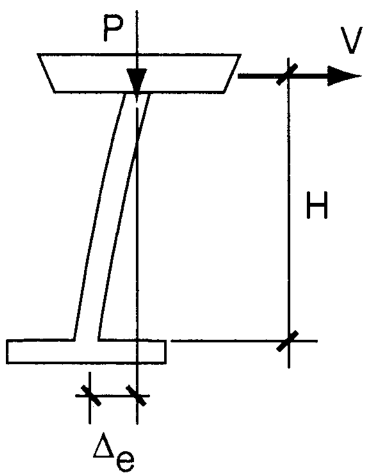
Consequences of Allowing Yielding in Structures

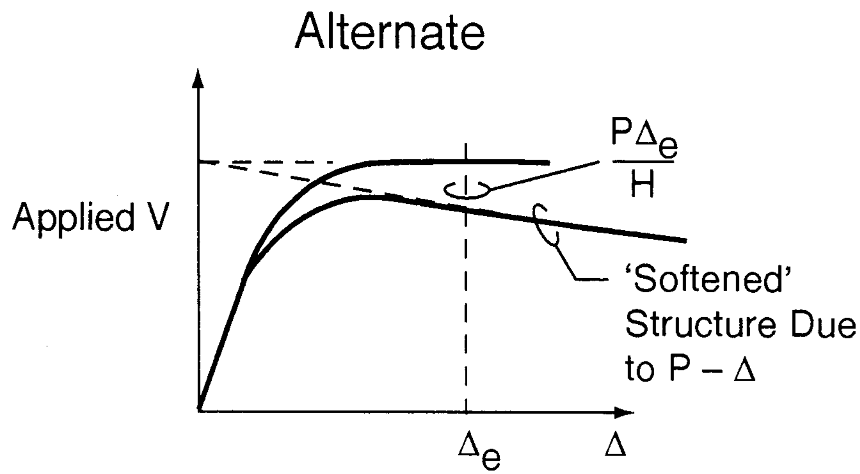
Implications:

- Ductility Demand μ Increases as Resistance F Decreases
- As μ Increases, Chance of Damage Increases
- As μ Increases, Special Detailing Becomes Necessary

Slenderness and P - Δ Effects

- AASHTO — Use Division I Method (Elastic Theory)
- Alternate — Increase Design Moment by $P \cdot \Delta_e$ to Account for Loss of Resistance at Δ_e (Concrete)

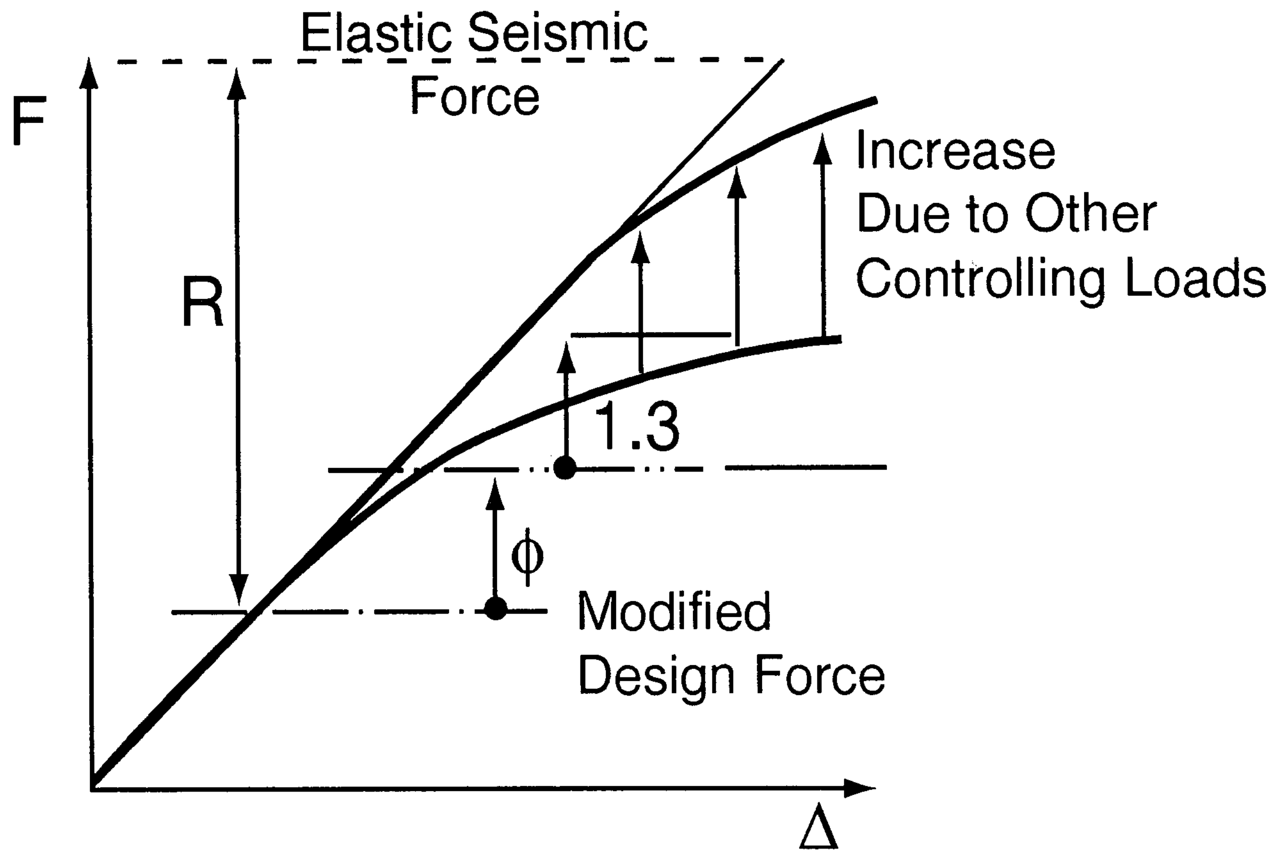




Non-Seismic Controlling Load Cases

- More Common in Lower Acceleration Zones
- Reduces Ductility Demands for Design Ground Motion
- May Significantly Increase Plastic Hinging Shear
- Foundation Sizes May Be Quite Large for Plastic Forces

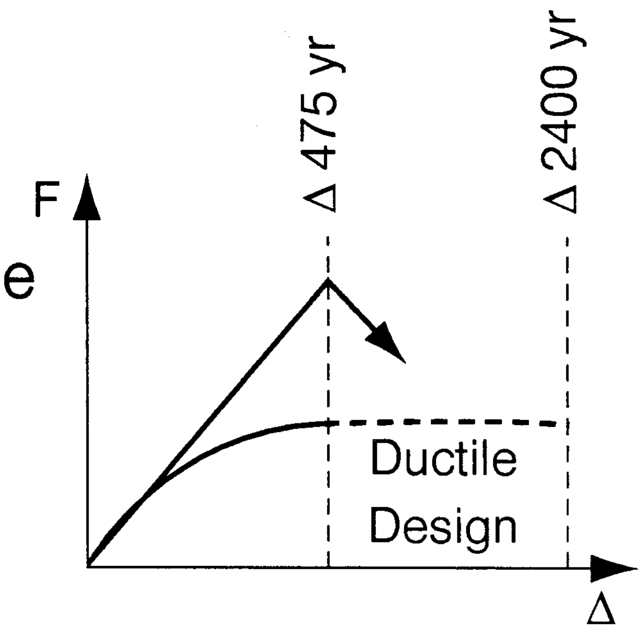
Recall F vs. Δ Behavior



Fail-Safe Issues

- Recall that Design Shaking
  = 10% Chance of Exceedence in 50 Years
  ≠ 10% Chance of Being Equal to Acceleration Level

- Provide for Ductile Response Up to and **Beyond** Design Ground Motion



Fail-Safe Issues (continued)

Spectral Acceleration for T = 0.3 sec
(Not Peak Ground Acceleration)

Ground Motion	San Francisco California		Boston Massachusetts	
475 Year Return Period (10% Chance of Exceedence in 50 Years)	1.75 g	1.7x	0.37 g	2.4x
2400 Year Return Period (2% Chance of Exceedence in 50 Years)	3.00 g		0.88 g	

Session 7
Column and Pier Design

- Intended Seismic Behavior
- **SPC B vs. SPC C and D Requirements**
- Wall Pier Design
- General Detailing Issues

Force Requirements; SPC B vs. Higher Categories

Design Forces	SPC B	SPC C and D
Column Flexure	$\frac{\text{Elastic}}{R}$	$\frac{\text{Elastic}}{R}$
Column Shear and Axial, Connections	$\frac{\text{Elastic}}{R}$	Plastic Hinging Forces, or Full Elastic Forces (Seismic)
Foundations	$\frac{\text{Elastic}}{R/2}$ Attempts to Force Column to Yield	Plastic Hinging Forces, or Full Elastic Forces (Seismic)

SPC B – Column Design

Consider Our Example Bridge / A= 0.15 g   3 ft Diameter Column

Column Design Forces:

M_u = 891 kip ft
P_u = 1049 kip
V_u = 58 kip

All Based on R = 5

8 #10 (1.00%)
ϕ = 0.7 for SPC B (Instead of ϕ = 0.5 SPC C and D)

Plastic Hinging Forces:

M_p = 1794 kip ft
V_p = 142 kip

Not Required in SPC B

SPC B – Column Design (continued)

Implications:

Flexure $M_p = 1794$ kip ft $< M_{LC1 + DL}^{elastic} = 3981$ kip ft $\therefore$ Column Yields

Shear $V_p = 142$ kip $> V_u = 58$ kip $\therefore$ Problem?

$\phi V_n = 135$ kip with Minimum Steel $\therefore$ Close, but ...

Is It Wise to Divide Column Shear by R in SPC B?

SPC B – Footing Design

Footing Design Forces:

$$\left.\begin{array}{l} M_u = 1497 \text{ kip ft} \\ V_u = 116 \text{ kip} \end{array}\right\} \text{All Based on } R = \frac{5}{2} = 2.5$$

Implications:

Rocking $M_p = 1794$ kip ft $> M_u = 1497$ kip ft (Transferred from Column) $\therefore$ Greater than 1/2 Uplift ? Footing Shear Problems ?

Sliding $V_p = 142$ kip $> V_u = 116$ kip Sliding Possible ?

General Statements – SPC B Design

Column Shear: Actual Shear Capacities May Be Much Less than Plastic Shear Demand

$\therefore$ Use R = 1 for Shear

or Use Procedure for SPC C and D

Footing Forces: Use of $\frac{R}{2}$ Works Reasonably Well for Footing Design **if** Seismic Loads Control

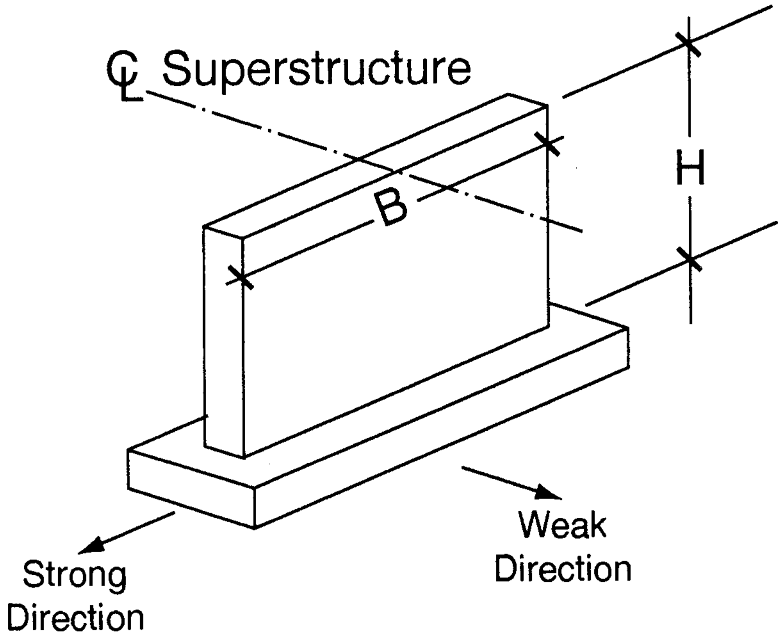
Other SPC B vs. Higher Category Issues

Issues	SPC B	SPC C and D
Seat Width	$N_B = 0.67 \cdot N_{C\,\&\,D}$	$N = (12'' + 0.03L + 0.12H) \cdot (0.000125S^2)$
Hinge Zone Confinement	Maximum s = 6"	Maximum s = 4"
Column Connection Shear Stress	NA	$v \leq 12\sqrt{f'_c}$ Normal Weight Concrete
Wall Pier Shear Stress	NA	$v \leq 2\sqrt{f'_c} + \rho_h f_y \leq 8\sqrt{f'_c}$
Restrainers	NA	Required Between Structure Sections

Session 7
Column and Pier Design

- Intended Seismic Behavior
- SPC B vs. SPC C and D Requirements
- **Wall Pier Design**
- General Detailing Issues

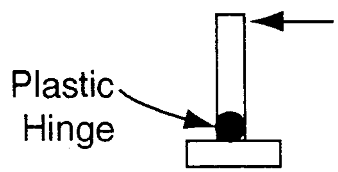
Wall Pier Design



Pier: $\frac{H}{B} < 2.5$ and Column: $\frac{H}{B} \geq 2.5$

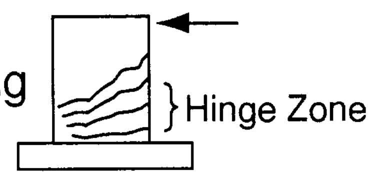
Design as Pier (or Wall), or
Design as a Column

Can Develop Plastic Hinging at Base of Wall



Design as Pier (or Wall)

Probably No Plastic Hinging at Base of Wall



Wall Pier Design (continued)

- **SPC B**

Strong Direction
$R = 2$

Weak Direction
$R = 2$ – Wall
or $R = 3$ – Column $\left\{ \begin{array}{l} \text{Meet SPC B} \\ \text{Confinement Requirements} \end{array} \right.$

Wall Pier Design (continued)

- **SPC C and D**

Same as B Plus:

Column – R = 3	• Meet SPC C and D Confinement • Design for Plastic Hinging • ϕ=0.5 • Minimum Column Steel, 1% or Arch.
Wall – R = 2	• Column Confinement Not Required • No Plastic Hinging Design • ϕ=0.7 • Minimum Horizontal /Vertical Steel Ratios • Limiting Shear Stress

Minimum Reinforcement for Wall Piers

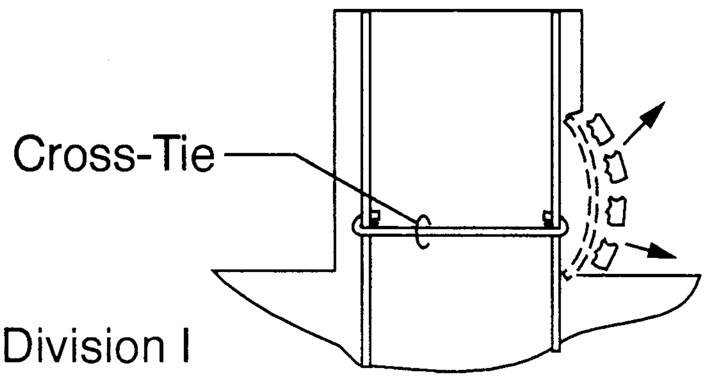
- Is an Issue When Non-Seismic Loads Control
- SPC B, What to Use?

Recommend if Local Requirements Are Less

- Use SPC C and D Values $\rho_h = \rho_v = 0.0025$
- ACI 318 $\rho_h = 0.0025$ $\rho_v = 0.0015$
- Local Agency / Durability Issue Mainly

Cross-Ties in Wall Piers

- **Design as Column** ⟶ Confinement Ratios Control
- **Design as Wall** ⟶ No Specific Criteria in Division I-A (SPC B or SPC C and D)
- **Purpose**

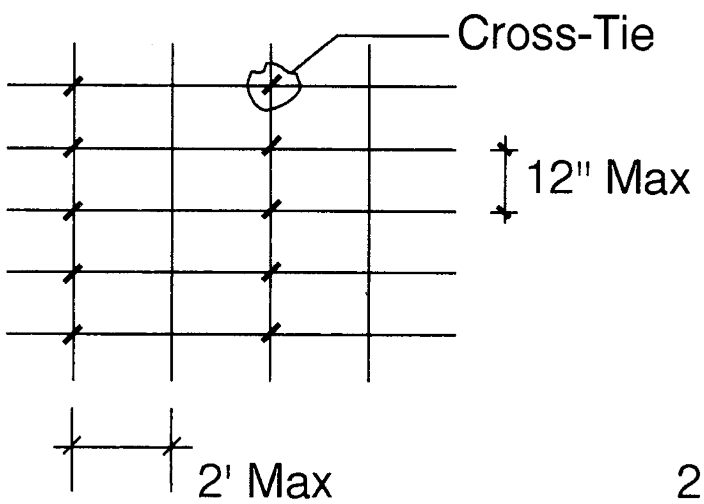


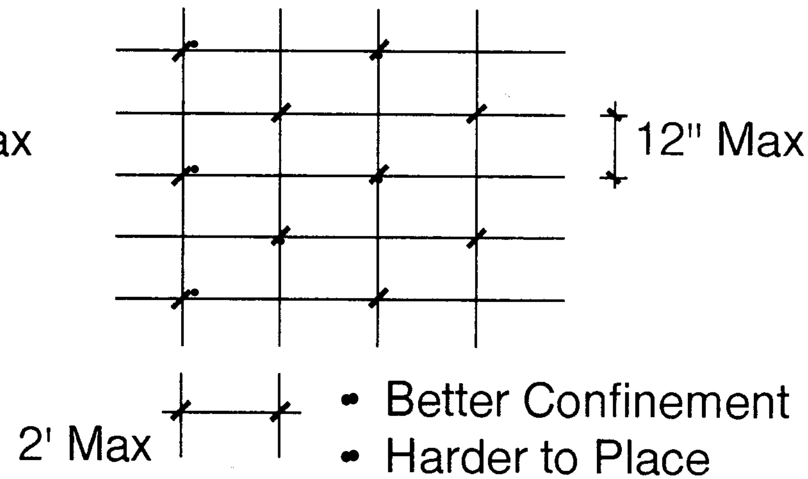
1. Cover Spalls
2. Vertical Bars Tend to Buckle
3. Cross Ties Restrain Vertical Bar

- **Options** – Division I
  – Caltrans

'Division 1' Cross-Ties

- Spacing of Ties s ≤ Least Member Dimension or 12 Inches
- Longitudinal Bars ≤ 2 Feet from a Restrained Bar





- Better Confinement
- Harder to Place Concrete

Elevation of Wall Steel

Session 7
Column and Pier Design

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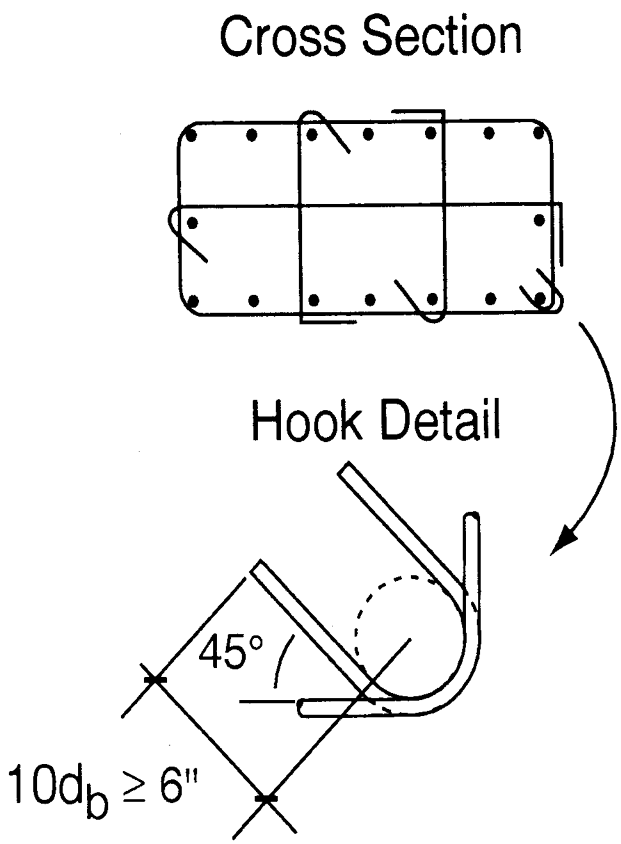
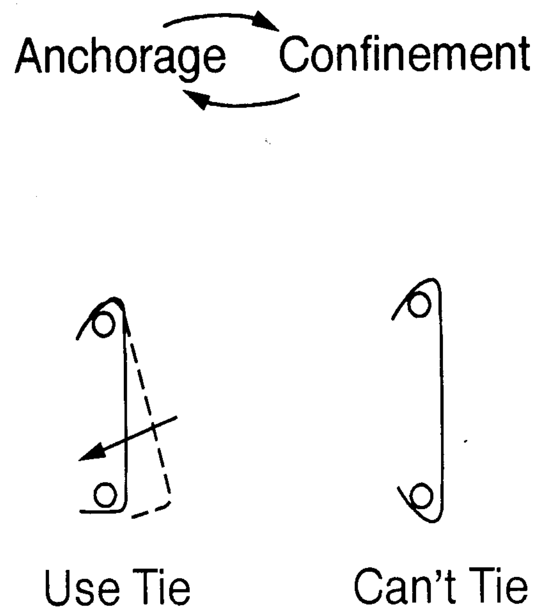
- Wall Pier Design
- **General Detailing Issues**

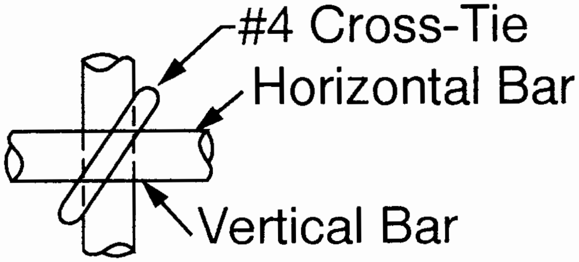
Tied-Column Details





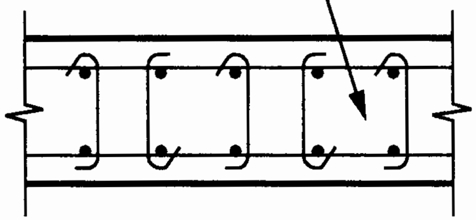
Cross-Ties in Walls

- **Tie Crosses Both Bars**

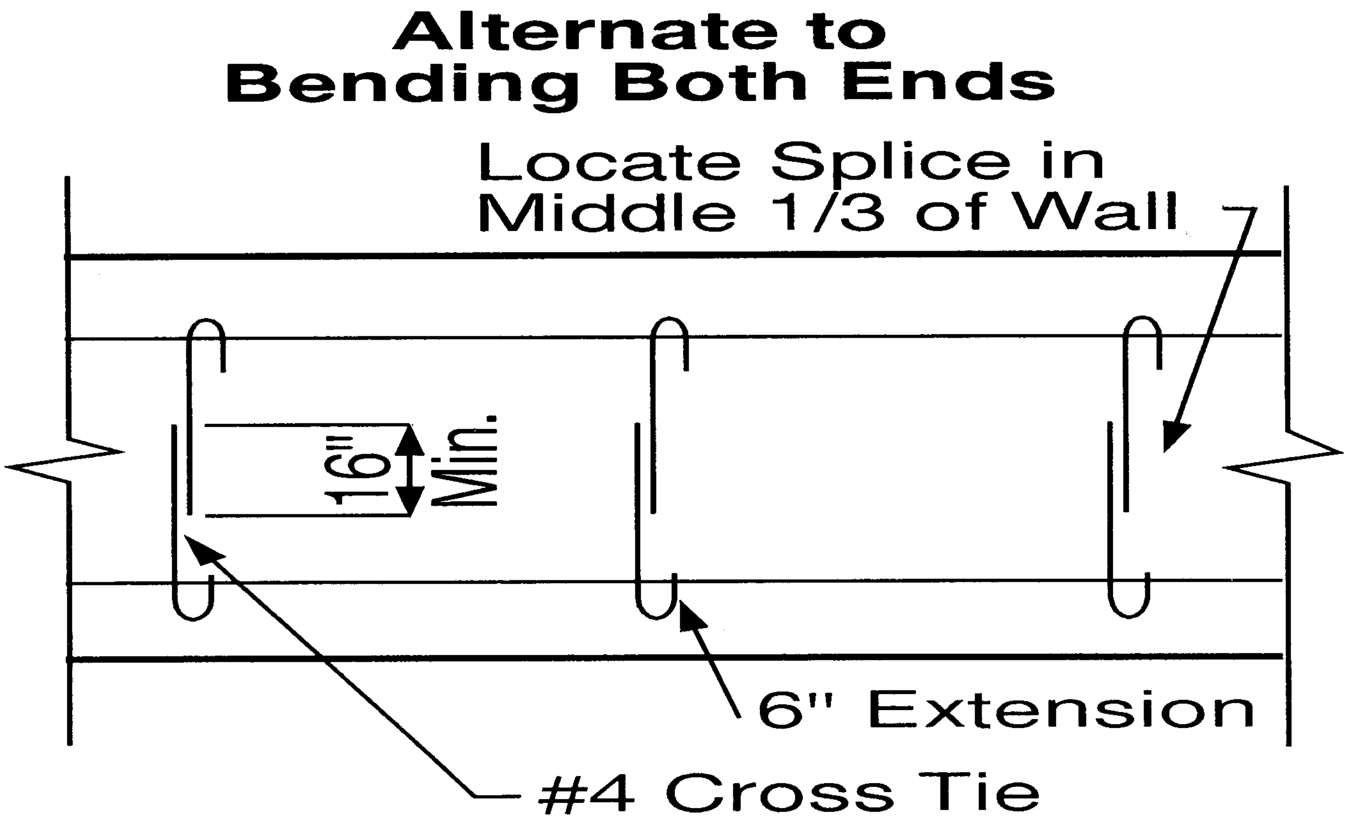


- **Alternate Cross-Tie 90° Bends**

Hooks of Adjacent Cross-Ties Face Each Other to Provide Space for Placing Concrete

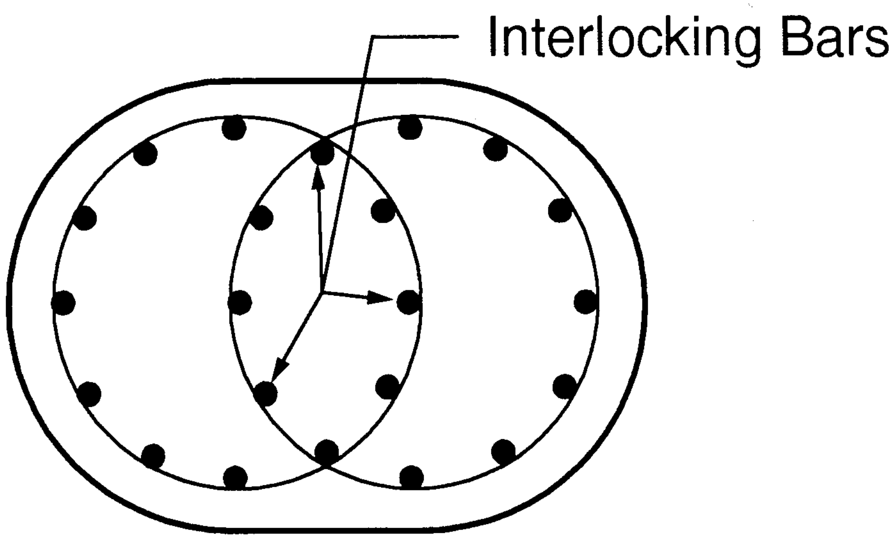


Cross-Ties in Walls (continued)



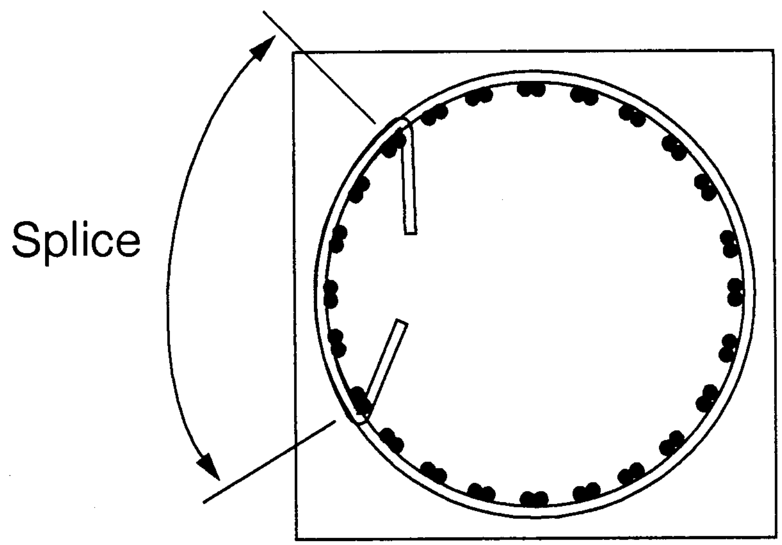
Interlocking Spiral

- Spirals Provide Improved Confinement for Rectangular Columns
- Cross-Ties May Be Required for Shear Strength



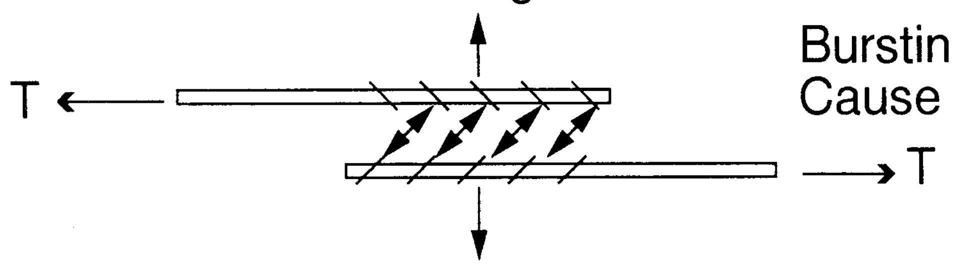
Spiral Splices

- Hooks Shall Be Placed to Avoid Vertical Reinforcement
- Lap Splices Not Permitted in End Regions
- Alternate: Weld Splice (A706 Steel)

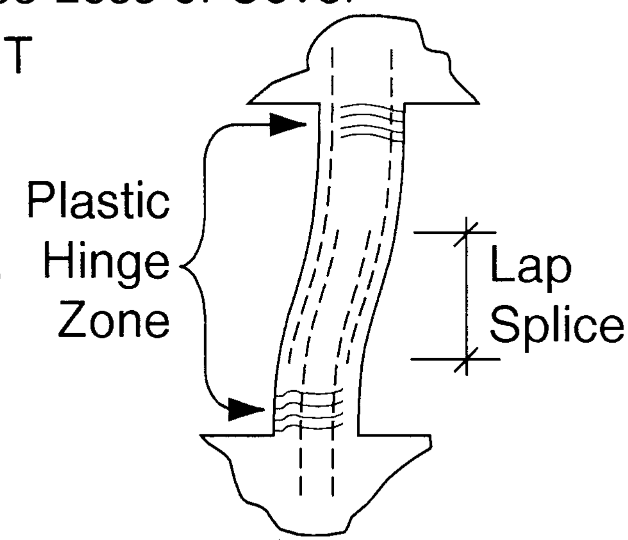


Reinforcement Splices

- Don't Splice Longitudinal Steel in Plastic Hinge Zones

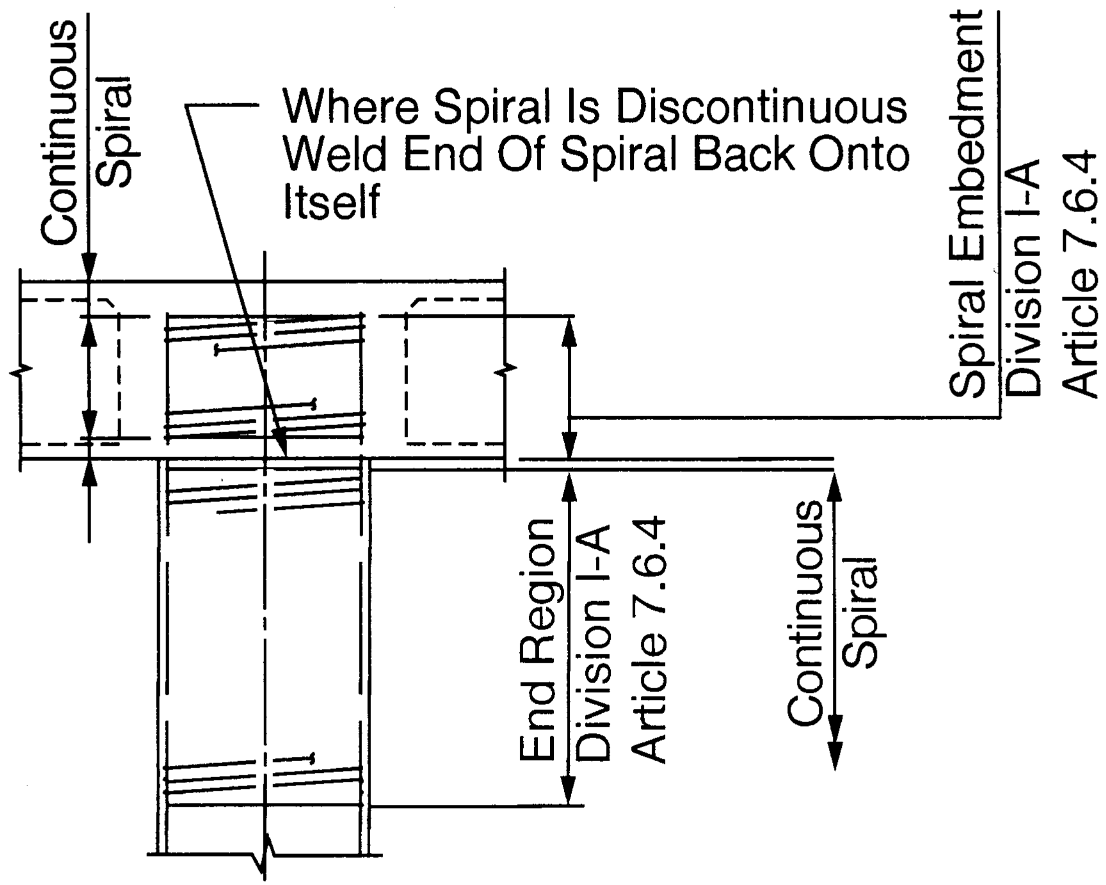


Bursting Forces Cause Loss of Cover

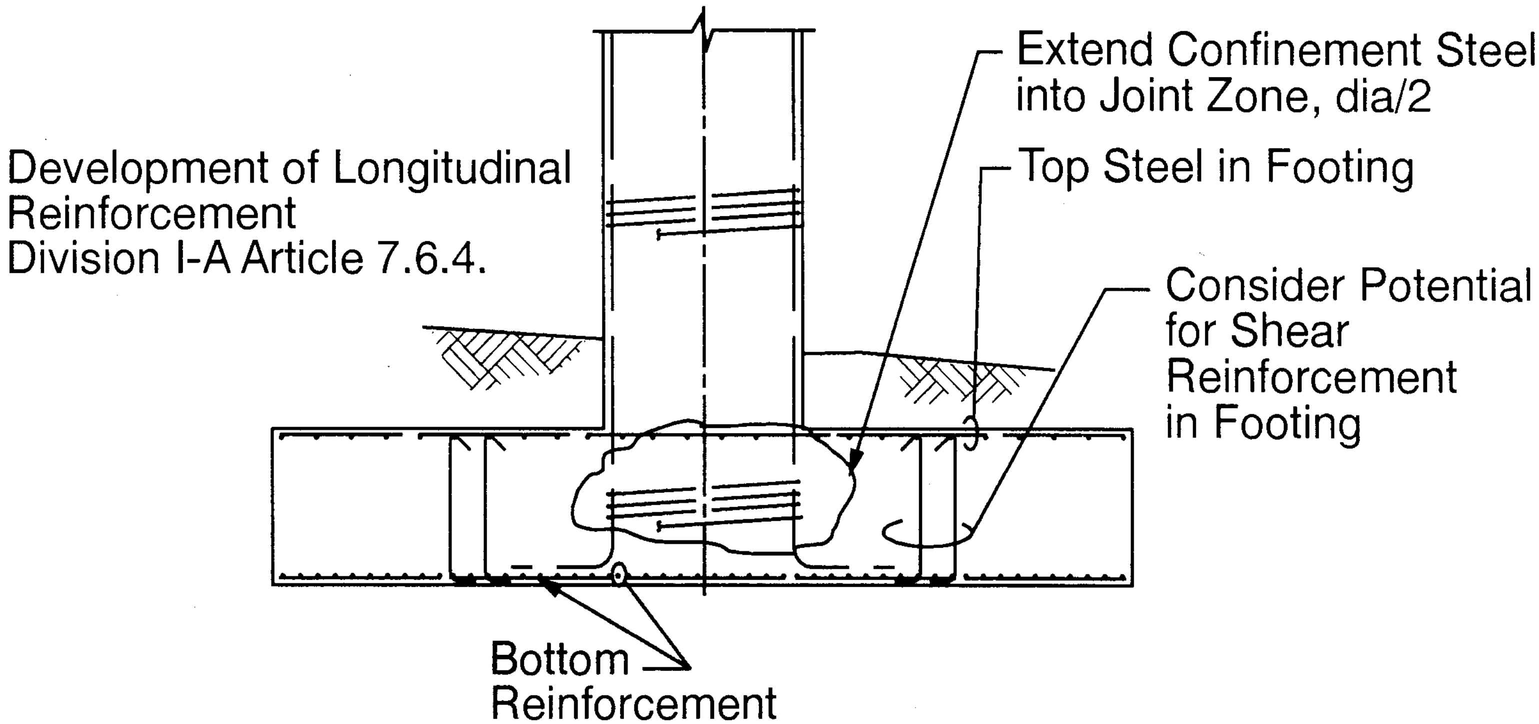


- Lap Splice in Middle Half of Columns (Low Moment Zone)
- Alternate: Welds or Mechanical Connectors (Staggered 24")

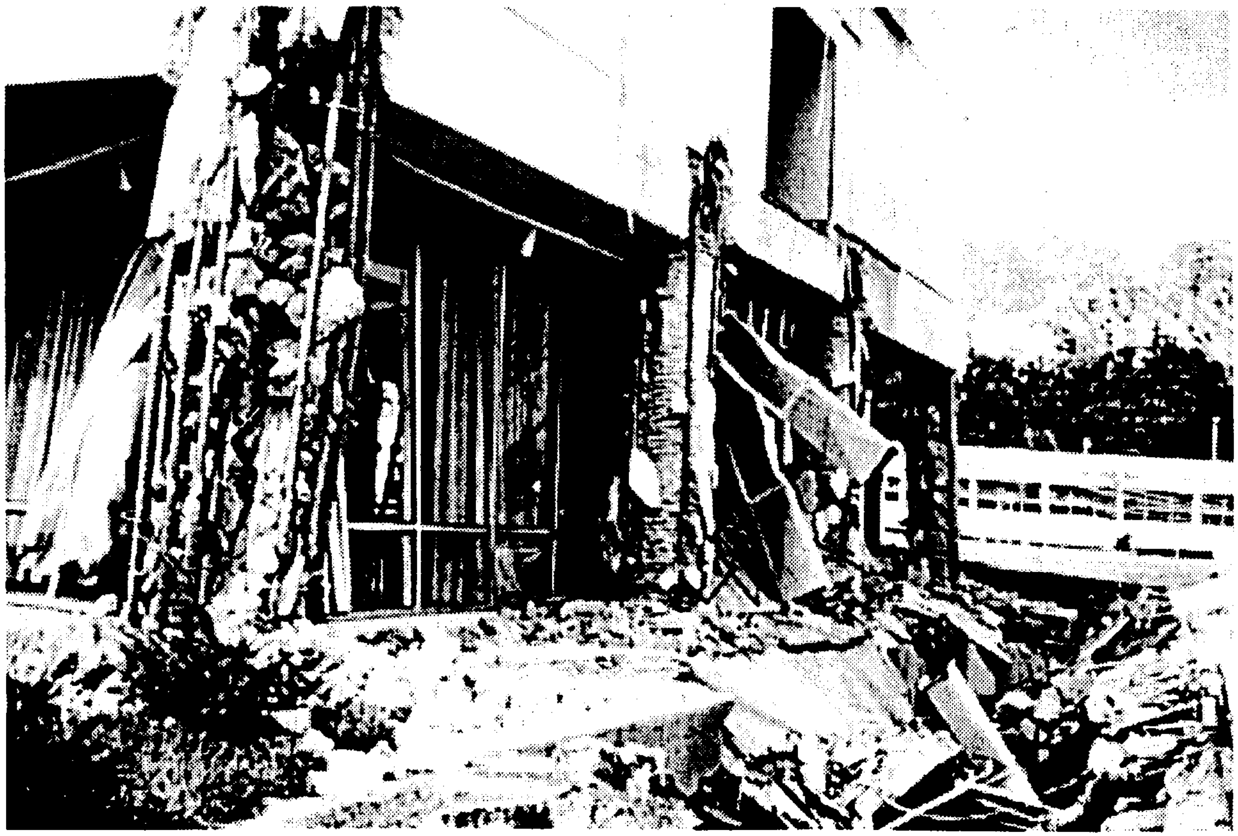
Connection at Cap Beam



Connection at Footings



Importance of Details



Session 7
Column and Pier Design

- **Questions and Answers**